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REPORT ON

TWO-SPAN STONE ARCHES BRIDGE NO. 114
US ROUTE 7 OVER THE NESHOBE RIVER
BRANDON, VERMONT

by Haley & Aldrich, Inc.
Bedford, New Hampshire

for CLD Consulting Engineers
Manchester, New Hampshire

File No. 41107-200
August 2017

HALEY & ALDRICH, INC.
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Revised 9 August 2017
August 2017
File No. 41107-200

CLD Consulting Engineers, Inc.
540 North Commercial Street
Manchester, New Hampshire 03101

Attention: John Byatt, P.E.

Subject: Two-Span Stone Arches Bridge No. 114
US Route 7 over the Neshobe River
Brandon, Vermont

Dear Mr. Byatt,

This geotechnical report summarizes the results of the geotechnical subsurface explorations and geotechnical laboratory testing, and provides foundation and construction recommendations for the design of the proposed abutments and retaining walls. Haley & Aldrich's services were completed in accordance with our proposal dated 18 February 2016, and your subsequent authorization.

We greatly appreciate the opportunity to be on your project team. Please do not hesitate to contact us if you wish to discuss the contents of this report or any aspect of the project.

Sincerely yours,
HALEY & ALDRICH, INC.

Meghan Brassard

Meghan M. Brassard
Senior Geotechnical Engineer

John G. DiGenova, P.E.
Associate

Enclosures

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1. Introduction

1.1 SITE LOCATION AND SURFACE CONDITIONS

The existing bridge No. 114 is located in the center of the Town of Brandon, Vermont and carries US Route 7 over the Neshobe River as shown on Figure 1 – Project Locus. US Route 7 at the bridge location is situated east/west and the Neshobe River is situated north/south. The area surrounding the bridge is in downtown Brandon and includes public and private businesses including the Brandon Town Hall, banks, cafes, and shops. Bedrock outcrops in the river are visible at the ground surface. A public park known as Green Park which includes a stone bench and gazebo is located to the southeast of the bridge.

1.2 EXISTING BRIDGE CONDITION

The existing bridge structure is a twin stone arch spanning the Neshobe River. The stone arches are approximately 12.8 m [42 ft] long with a 2 m [6.6 ft] long concrete extension consisting of concrete abutments, wingwall, and pier on its southern/downstream end. The concrete extension supports a 1.5 m [5 ft] concrete sidewalk slab with concrete parapet. Each arch has a span of approximately 5.2 m [17.1 ft] from springline to springline. The structure extends underneath US Route 7 before ending at the approximate location of the northern sidewalk. The bridge is skewed on a 15-degree angle.

In 2011, emergency repairs were performed on the east barrel of the arch structure following extensive flooding caused by Tropical Storm Irene. The repairs consisted of repointing and pressure grouting the arch stone, including mortar injections from above the arch to fill voids, as well as repairs to the downstream stone facing of arches. It is our understanding that only the first 2.1 to 3 m [7 to 10 ft] of the east barrel was repaired. There are concerns with the current condition of the arch structure of the bridge which include significant mortar loss, water infiltration and intrusion through the arch stones, and poor condition of the concrete extensions which has created water infiltration holes along the concrete curb.

1.3 PROPOSED CONSTRUCTION

We understand the proposed construction will include rehabilitation of the existing bridge double arch structure, replacement of the existing sidewalk bridge, replacement of two retaining walls, and other related roadway work. The two new retaining walls to be replaced are on the west and east side of the sidewalk bridge and will be situated in the same footprint as the existing retaining walls with a footing width of about 2.5 m [8.2 ft] and footing length of about 2.8 m [9.2 ft] for the west wall and footing length of about 13 m [42.6 ft] for the east wall. The new east and west retaining walls will have bottom of footings at El. 124.25 and El. 125, respectively. Based on the plan set titled "Proposed Improvement Bridge Project, Town of Brandon, County of Rutland, Us Route 7 (Principal Arterial) Bridge No. 114" by CLD Consulting Engineers, the two abutments (Abutment 1 on the east side and Abutment 2 on the west side) for the new sidewalk bridge will bear on bedrock with the bottom of footing at El. 122.8 and 123, respectively. We understand based on discussions with CLD the east retaining wall footings will be situated at a depth of 1.5 m [5 ft] below grade and the west retaining wall footings will be situated a depth of 0.9 m [3 ft] below grade.

Based on information provided by CLD, we understand the preliminary bearing pressures for the structures are as follows:

Structure	Strength Bearing Pressure	Strength Eccentricity	Service Bearing Pressure	Service Eccentricity
Abutments 1	1132 kPa [23.6 ksf]	0.85 m [2.8 ft]	802 kPa [16.8 ksf]	0.85 m [2.8 ft]
Abutment 2	717 kPa [15 ksf]	0.71 m [2.3 ft]	507 kPa [10.6 ksf]	0.72 m [2.4 ft]
East Retaining Wall	22 kPa [0.46 ksf]	0.19 m [0.6 ft]	15 kPa [0.31 ksf]	0.11 m [0.4 ft]
West Retaining Wall	13 kPa [0.27 ksf]	0.03 m [0.1 ft]	10 kPa [0.21 ksf]	0.01 m [0.03 ft]

We note that the bearing material of the retaining walls is unknown. The condition of the existing retaining walls shows little signs of settlement. It is unknown whether these retaining walls have been repaired in the past. We have assumed the bearing material consists of medium dense granular soil or better. The bearing conditions must be confirmed during construction.

2. Subsurface Explorations and Testing

2.1 TEST BORINGS

A drilling program including 5 test borings was completed during the period 4 to 6 August 2015. The test borings were completed by New England Boring Contractors, Inc. of Derry, New Hampshire. A lane closure and traffic control was provided by Green Mountain Flagging under contract to New England Boring Contractors, Inc. The traffic control plan prepared by CLD was utilized for the lane closure.

A representative of Haley & Aldrich was present to document subsurface conditions. The test borings included 4 borings (HA-B1, HA-B1A, HA-B1B, and HA-B1C) at the area of the existing southeast abutment and 1 boring (HA-B2) at the area for the proposed future retaining wall. Locations of the test borings are presented on Figure 2.

Four attempts were made in the area of boring HA-B1 to advance the boring to natural materials but due to the presence of cobbles, boulders, granite blocks, or other obstructions the boring could not be advanced beyond 5.4 m [17.7 ft]. It is uncertain if the boring encountered natural materials or not. The borings ranged in depth from 1.4 to 5.4 m [4.6 to 17.7 ft] below existing grades.

The test boring reports are presented in Appendix A.

2.2 TEST PROBES

A series of roadway probes was completed during the period of 3 to 5 August 2015. The test probes were completed by New England Boring Contractors, Inc. of Derry, New Hampshire. A lane closure and traffic control was provided by Green Mountain Flagging under contract to New England Boring Contractors, Inc. The traffic control plan by CLD was utilized for the lane closure. A representative of Haley & Aldrich was present to document subsurface conditions. The roadway probes included 10 locations (HA-P1 through HA-P10 Alt. excluding HA-P8 and HA-P10) within the roadway area. Three probe locations (HA-P8, HA-P10, and HA-P11) were eliminated prior to the start of drilling after conversations with CLD. Locations of the roadway probes are presented on Figure 2. The roadway probes extended through the pavement asphalt, concrete roadway slab (if encountered) and were terminated in the underlying soils, with the exception of HA-P3 which was performed to identify the asphalt thickness only. The probes ranged in depth from 0.2 to 1.2 m [0.66 to 4 ft] below existing grades.

The roadway probes are presented in Appendix A.

2.3 GEOTECHNICAL LABORATORY TESTING

Laboratory grain-size (ASTM D 422) analyses were performed on 8 soil sample recovered from the roadway probes beneath the roadway and/or roadway slab. The geotechnical laboratory testing was completed by GeoTesting Express, Inc. of Acton, Massachusetts. The results of the soil laboratory testing are presented in Appendix B.

3. Subsurface Conditions

3.1 SOIL CONDITIONS

The test borings and roadway probes encountered the following generalized soil strata at the site, in order from increasing depth below ground surface. Some strata may be missing at particular locations. We note that the roadway probes were extended just below the roadway/roadway-slab and terminated in fill soils 0.2 to 1.2 m [0.6 to 4 ft] below ground surface, and other soil information below this depth for the area is based on the test borings which were extended to bedrock.

Asphalt: A thin layer of asphalt, ranging in thickness from 0.09 to 0.15 m [0.3 to 0.5 ft], was encountered at each exploration location at the ground surface.

Roadway Slab: The concrete roadway slab was encountered below the asphalt at 7 of the exploration locations. The roadway slab was not observed at locations HA-P1, HA-P2, HA-B1, HA-B1A, HA-B1B, HA-B1C, and HA-B2. The roadway slab was cored at each location where encountered except HA-P3. At the locations where slab was cored it ranged from 0.15 to 0.19 m [0.5 to 0.6 ft] thick. Three locations (HA-P4, HA-P9, and HA-P10 Alt.) encountered steel reinforcement at multiple depths along the cores. The steel reinforcement appeared to be about 10 mm diameter (close to a No. 3 size rebar).

Fill: The fill encountered beneath the asphalt and/or roadway slab from the probes was described as medium dense to dense well graded SAND (SW), poorly graded SAND (SP), poorly graded SAND with silt (SP-SM), well graded GRAVEL with silt (GW-GM), poorly graded GRAVEL with silt (GP-GM), poorly graded GRAVEL (GP), and/or silty SAND (SM) with varying amounts of gravel, sand, and silt. The fill was not fully penetrated at the roadway probe locations. The probes extended to depths ranging from 0.2 to 1.2 m [0.7 to 4 ft] below the pavement surface.

The fill encountered beneath the asphalt at the boring locations was described as medium dense to dense poorly graded SAND (SP), silty SAND (SM), and/or poorly graded GRAVEL (GP) with varying amounts of sand, gravel, and silt. The fill was most likely fully penetrated at locations HA-B1B and HA-B2 where it was found to be 1.6 and 2.4 m [5.2 to 7.9 ft] thick, respectively.

A large number of obstructions were observed in the fill at locations HA-B1, HA-B1A, HA-B1B, and HA-B1C. The obstructions were observed from 1.2 to 3.8 m [3.9 to 12.5 ft] below the pavement surface and contained possible boulders, granite blocks, steel plates, etc. that the drilling equipment was not able to advance through. A loss of water return was also noted during the drilling through the fill at these locations. The fill in this area also contained possible brick, ceramic, glass, wood chips, and saw dust at location HA-B1B. A pocket of ORGANIC SOILS (OL/OH) was encountered from 4.3 to 5.2 m [14.1 ft to 17.1 ft] below the pavement surface at location HA-B1B just above the probable top of bedrock. The fill was not fully penetrated at locations HA-B1, HA-B1A, and HA-B1C due to the presence of obstructions.

Forest Mat: A possible old forest mat or topsoil layer was encountered at a depth of 1.5 m [4.9 ft] (El. 124.8) below the ground surface at boring location HA-B2. The layer was 1.1 m thick and consisted of very loose to loose dark brown ORGANIC SOILS (OL/OH) and silty SAND (SM) and was directly above the bedrock.

Bedrock: Probable bedrock was encountered at 2 test borings (HA-B1B and HA-B2) at depths of 5.2 and 2.6 m [17.1 and 8.5 ft] below ground surface (El. 121.2 and 123.7 respectively). The roller bit was advanced 0.2 to 0.5 m [0.7 to 1.6 ft] into the probable bedrock to confirm the bedrock presence.

Based on published information for Brandon, Vermont from the United States Geological Survey (USGS), the bedrock is most likely quartzite from the Danby and Potsdam Formation or dolomite from the Gorge Formation.

Groundwater was not observed at the roadway probes or boring locations. The Neshobe River level just below the bridge is at about El. 122.7 (normal river level).

3.2 GROUNDWATER

Groundwater was observed at seven boring locations (HA-2 through HA-7, and HA-10) at depths ranging from about 0.6 to 12.3 m [2.0 to 40 ft ft] below ground surface. Mottling was observed between depths 0.6 to 3.7 m [2 to 12.1 ft] at some boring locations, indicating a perched water level throughout the site. Water levels can be expected to vary with seasonal changes, precipitation, snow melt, construction activities, and other factors. Water levels encountered during and following construction may differ from those encountered in the explorations.

4. Geotechnical Design Recommendations

4.1 GENERAL

Geotechnical design recommendations for the subject project were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Seventh Edition, 2014;
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, Second Edition, 2011 with 2012 Interim Revisions; and
- VTrans Structures Design Manual, 5th Edition, 2010

4.2 FOUNDATION DESIGN RECOMMENDATIONS

Our assessment for all the foundation structures included strength, service, and extreme event limit state bearing resistance, sliding, and global stability calculations and provides results for LRFD guidelines. Our assessment is based on elevations, loading, and geometry provided by CLD. We note that the abutment foundations will bear on bedrock and the retaining wall foundations are assumed to bear on medium dense granular soil. We also note that the recommendations presented below assume that the retaining walls will be situated in the same footprint as the existing retaining walls.

The evaluations for the abutments and retaining wall are provided in Appendix C.

Abutments 1 and 2

- A nominal strength limit state bearing resistance of about 2633 kPa [55 ksf] per Section 10.6.3.2. A resistance factor of 0.45 is recommended per Table 10.5.5.2.2-1for a factored bearing resistance of about 1197 KPa [25 ksf].
- A nominal service limit state bearing resistance of about 1915 kPa [40 ksf] for about 25 mm [1 in.] of elastic settlement per Section 10.6.2.6.1. A resistance factor of 1.0 is assumed for LRFD for the service limit state.
- A nominal extreme event limit state bearing resistance of about 2633 kPa [55 ksf]. A resistance factor of 0.8 is recommended in accordance with LRFD for a factored bearing resistance of about 2106 kPa [44 ksf].
- The abutments should be assessed to verify that they are capable of resisting an unfactored load due to total lateral earth pressure. An active earth pressure coefficient, Ka, of 0.31 is recommended for use with an assumed backfill soil friction angle of 32 degrees. A total unit weight of 18.8 kN/m^3 [120 pcf] should be used for the backfill. This assumes that the back of the wall is fully drained and hydrostatic pressure is not allowed to build up behind the wall.

For surcharges on walls, the resulting lateral loads should be calculated based on a uniform lateral pressure equal to 0.3 times the vertical surcharge pressure acting on the backfilled side of the wall (active pressure), applied over the full height of the wall. The vertical surcharge pressure should be calculated in accordance with AASHTO LRFD Table 3.11.6.4-1 and using a soil unit weight of 18.8 kN/m^3 [120 pcf].

- A coefficient of friction between the concrete footing base and the bedrock of 0.7 per Table 3.11.5.3-1 may be used to calculate the nominal sliding resistance. A resistance factor of 0.8 is recommended to calculate shear sliding resistance per Table 10.5.5.2.2-1.

- We recommend a combination of bedrock keys and use of steel dowels grouted into bedrock be used to increase resistance to sliding on rock surfaces sloping at an angle greater than 4H:1V. The keys and dowels would also be used to resist hydrodynamic and ice loads. Details for dowels will be developed during construction depending on the configuration of the bedrock surface encountered at the time of bearing surface preparation. At a minimum, No. 8 bars should be embedded at least 0.6 m [2 ft] into sloping or stepped bedrock bearing surfaces at a 1.2 m by 1.2 m [4 ft by 4 ft] plan spacing based on guidance in the VTrans Structures Design Manual.

- Based on the bedrock bearing material, by inspection, the global stability for the abutments is acceptable.

East and West Retaining Walls

- A nominal strength limit state bearing resistance of about 369 kPa [7.7 ksf] for the east wall and 297 kPa [6.2 ksf] for the west wall per Section 10.6.3.1. A resistance factor of 0.45 is recommended per Table 10.5.5.2.2-1for a factored bearing resistance of about 168 kPa [3.5 ksf] for the east wall and 134 kPa [2.8 ksf] for the west wall. This is based on the footing geometry and eccentricity discussed previously.

- A nominal service limit state bearing resistance of about 144 kPa [3 ksf] for the east and west walls for about 25 mm [1 in.] of elastic settlement per Section 10.6.2.6.1. A resistance factor of 1.0 is assumed for LRFD for the service limit state. This is based on the footing geometry and eccentricity discussed previously.

- A nominal extreme event limit state bearing resistance of about 369 kPa [7.7 ksf] for the east wall and 297 [6.2 ksf] for the west wall. A resistance factor of 0.8 is recommended in accordance with LRFD for a factored bearing resistance of about 297 kPa [6.2 ksf] for the east wall and 239 kPa [5 ksf] for the west wall.

- The retaining wall should be assessed to verify that it is capable of resisting an unfactored load due to total lateral earth pressure. An active earth pressure coefficient, Ka, of 0.31 is recommended for use with an assumed backfill soil friction angle of 32 degrees. A total unit weight of 18.8 kN/m^3 [120 pcf] should be used for the backfill. This assumes that the back of the wall is fully drained and hydrostatic pressure is not allowed to build up behind the wall.

  For surcharges on walls, the resulting lateral loads should be calculated based on a uniform lateral pressure equal to 0.3 times the vertical surcharge pressure acting on the backfilled side of the wall (active pressure), applied over the full height of the wall. The vertical surcharge pressure should be calculated in accordance with AASHTO LRFD Table 3.11.6.4-1 and using a soil unit weight of 18.8 kN/m^3 [120 pcf].

- A coefficient of friction between the cast-in-place concrete footing base and the medium dense granular backfill of 0.55 per Table 3.11.5.3-1 may be used to calculate the nominal sliding resistance. A resistance factor of 0.8 is recommended to calculate shear sliding resistance per Table 10.5.5.2.2-1.

- The global stability analysis for the east retaining wall assumed the geometry from the plan set provided to us by CLD. The calculated factor of safety for the service limit state was about 3.3 for the east wall. An acceptable resistance factor for a slope that contains or supports a structural element from LRFD Section 11.6.2.3 is 0.65 (equivalent to a factor of safety of 1.5). The calculated factor of safety for the extreme event limit state was about 2.4 for the east wall. An acceptable equivalent factor of safety for extreme event limit state calculations is assumed to be 1.1 for LRFD. The global stability of the west wall (shorter wall) is acceptable based on observation from the results of the east wall.

- Footings should bear a minimum 5 ft below finished grade for frost protection. If the full frost depth is not achievable, an alternative is to place insulating board around the footing for frost protection. We recommend using 1 in. of insulating board equivalent to about 1 ft of soil to achieve the minimum frost depth. The insulating board should be protected from hydrocarbons and animals.

4.3 SEISMIC DESIGN

Our recommendations with respect to seismic considerations for bridge design based on the subsurface conditions observed during the 2015 explorations are as follows:

- The site is classified as Site Class D in accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design.

- The Site-Specific Accelerations for the short 0.2 second period (S_{DS}) and long 1.0 second period (S_{D1}) in accordance with LRFD are assumed to be 0.277 g and 0.118 g, respectively.

- An acceleration coefficient, As, of 0.126 g is recommended in accordance with LRFD.

The seismic evaluation calculations are presented in Appendix C.

5. Construction Considerations

5.1 GENERAL

Construction of the proposed bridge abutments and retaining walls should be performed with the Design Specifications and the VTRANS Standard Specifications for Construction. The following sections provide additional comments on construction which are specifically relevant to the project.

5.2 PREPARATION OF ABUTMENT AND RETAINING WALL FOOTING SUBGRADES

Bedrock Subgrade (Abutments)

As noted previously, it is recommended that the rock bearing surfaces slope no steeper than an angle of 4H:1V. The abutment footing should be on clean sound bedrock per VTRANS Structures Design Manual. A Geotechnical Engineer should observe the exposed rock subgrade surface to confirm the surface is suitable for bearing. Based on guidance in the VTRANS Structures Design Manual, if necessary, step footing heights should be a minimum of 0.6 m [2 ft] or the footing thickness, whichever is greater.

Soil Subgrade (Retaining Walls)

The nearby borings indicated a layer of organic soils is present in the area of the retaining walls. During construction, if organics are encountered they should be removed. The organics should be removed within the Zone of Influence (ZOI) of the retaining wall footings. The ZOI is defined as the zone beneath footings and beneath imaginary lines extending from points 0.3048 m [1 ft] laterally beyond the footing outer bottom edges and out and down on a 1H:1V slope to the bearing soils. If overexcavation is necessary, we recommended Granular Backfill be used for backfill. The granular non-organic subgrade soils should be proof compacted with a minimum 9071 kg [10 ton] vibratory roller in the presence of the Geotechnical Engineer prior to placement of the footing to confirm adequate bearing.

5.3 OBSTRUCTIONS

Obstructions were encountered at some of the test boring locations in the existing fill. The obstructions included cobbles, boulders, possible granite blocks, and possible steel plates. If obstructions are encountered during construction, they should be removed to the extent required and any resulting voids backfilled.

5.4 BACKFILLING

Backfilling of abutments and the retaining walls will be required for construction of roadway embankments and for support of pavement sections. It is recommended that Granular Backfill be used for fill or backfill behind the abutments and retaining wall. Backfill should be compacted in accordance with VTRANS standards.

5.5 DEWATERING

If required, dewatering could be achieved using a system of sumps and pumps. It will be important to control groundwater and surface water to enable all final excavation and construction to be conducted in-the-dry. Care should be taken to maintain water at levels below the exposed soil and backfill subgrade at all times.

6. Closure

This report has been prepared for specific application to Bridge No. 114 in Brandon, Vermont, as understood by Haley & Aldrich at this time. In the event that changes in the design or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid unless they are reviewed and modified in writing by Haley & Aldrich. Our recommendations are based in part upon data obtained from the reference subsurface exploration program. The nature and extent of variation between explorations may not become evident until construction. If significant variations then appear, it may be necessary to re-evaluate the recommendations of this report.

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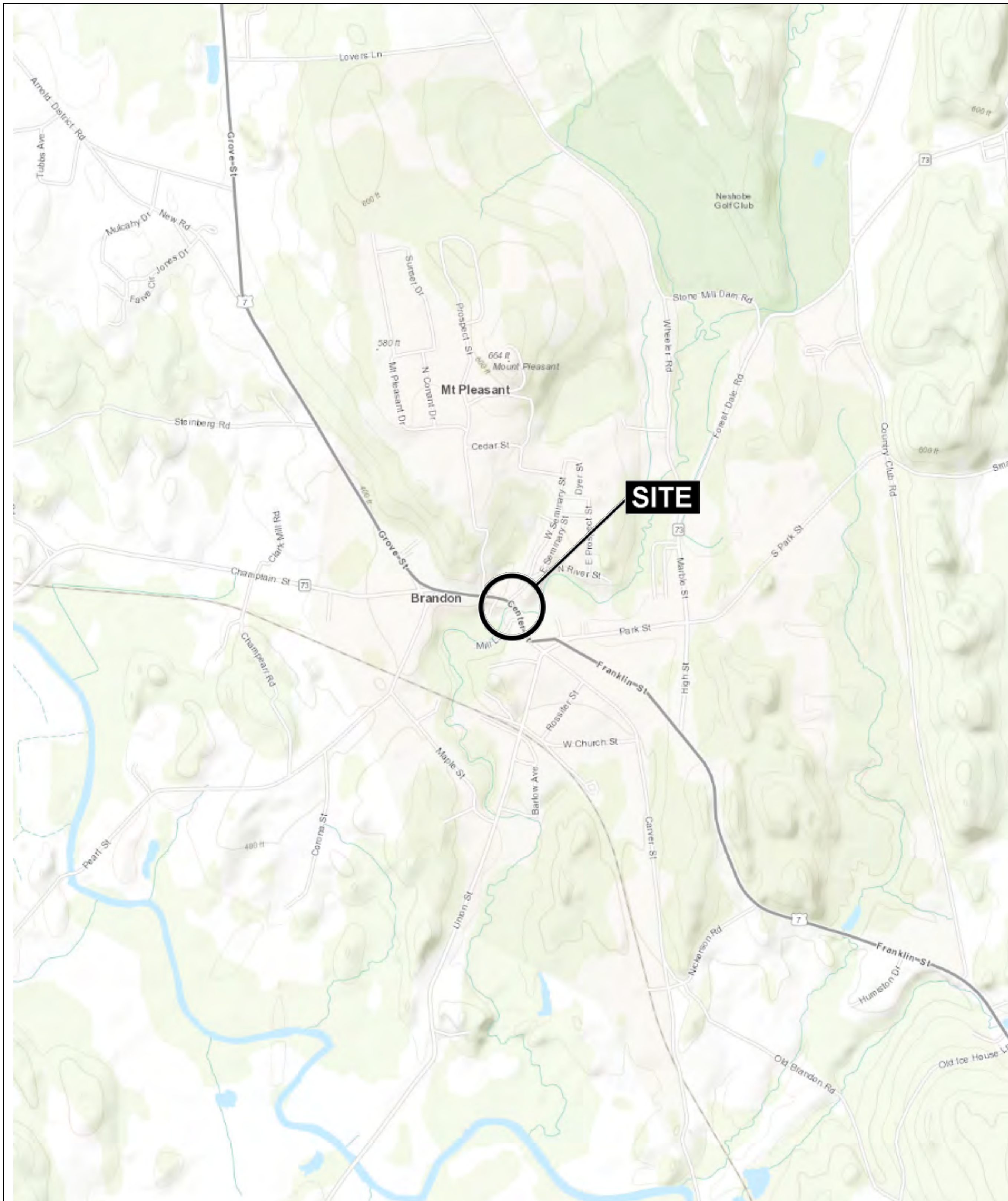
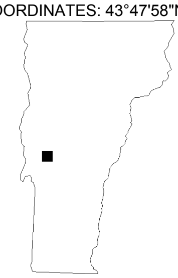


MAP SOURCE: ESRI

SITE COORDINATES: 43°47'58"N, 73°5'20"W



HALEY ALDRICH

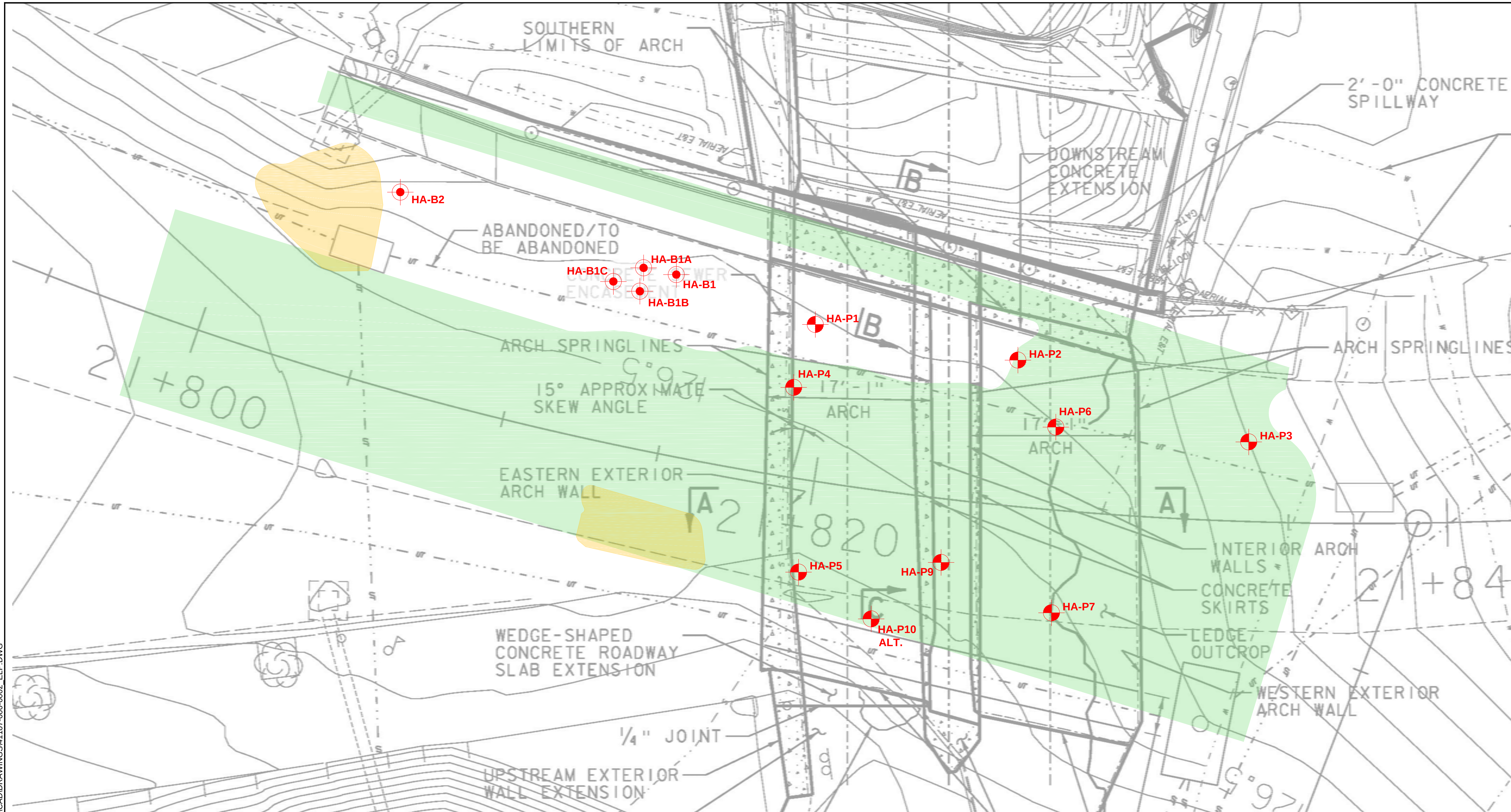
TWO-SPAN STONE ARCHES BRIDGE NO. 114
US ROUTE 7 OVER THE NESHOBE RIVER
BRANDON, VERMONT

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
JUNE 2017

FIGURE 1

41107-100_1_LOCUS.PDF



LEGEND

HA-B1 DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING COMPLETED BY NEW ENGLAND BORING CONTRACTORS, INC, DURING THE PEROD 4 TO 6 AUGUST 2015

HA-P4 DESIGNATION AND APPROXIMATE LOCATION OF TEST PROBE COMPLETED BY NEW ENGLAND BORING CONTRACTORS, INC. DURING THE PERIOD 3 TO 5 AUGUST 2015

APPROXIMATE LOCATION OF POSSIBLE DISTURBED SOILS IDENTIFIED IN GEOPHYSICAL REPORT BY HAGER-RICHTER DATED JULY 2015

APPROXIMATE LOCATION OF REINFORCED AREA IDENTIFIED IN GEOPHYSICAL REPORT BY HAGER-RICHTER DATED JULY 2015

NOTES

1. BASE PLAN WITH EXISTING GROUND SURFACE CONTOURS PROVIDED BY CLD CONSULTING ENGINEERS, INC. IN ELECTRONIC FORMAT ON 18 AUGUST 2015.
2. EXPLORATION LOCATIONS WERE LAID OUT IN THE FIELD BY A REPRESENTATIVE OF HALEY & ALDRICH BY PACING OR TAPING FROM EXISTING SITE FEATURES.
3. EXPLORATIONS WERE MONITORED IN THE FIELD BY A REPRESENTATIVE OF HALEY & ALDRICH.
4. ELEVATIONS ARE IN METERS AND REFERENCED THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).

0 10 20
SCALE IN FEET

HALEY ALDRICH

TWO-SPAN STONE ARCHES BRIDGE NO. 114
US ROUTE 7 OVER THE NESHOBE RIVER
BRANDON, VERMONT

EXPLORATION LOCATION PLAN

SCALE: AS SHOWN
JUNE 2017

FIGURE 2

LIBBY, PAT Printed: 9/3/2015 12:55 PM Layout: FIG-2
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APPENDIX A

Test Boring Logs and Probes

HALEY ALDRICH — TEST BORING REPORT

METRIC
Boring No. HA-B1

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	4 Aug 2015
Finish	4 Aug 2015
Driller	M. Thompson
H&A Rep.	M. Hatton
Elevation	126.40 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	HW	S	--
Inside Diameter (cm)	10.16	3.49	--
Hammer Weight (kg)	136.08	63.50	-
Hammer Fall (cm)	61.0	76.20	-

Drilling Equipment and Procedures
Rig Make & Model: Mobile B57 Track
Bit Type: Roller Bit
Drill Mud: None
Casing: HW Drive to 1.43 m
Hoist/Hammer: Winch, Automatic Hammer

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.31 0.09		Note: Used 12.7 cm roller bit to cut through asphalt. Note: 10.12 cm asphalt. -ASPHALT-										
	10 11 10 21	S1 30	0.30 0.91			SP	Note: Roller bit into sand and gravel to 0.30 m prior to sampling. Medium dense brown poorly graded SAND with gravel (SP), mps 2.54 cm, no structure, oil-like odor at top 5.08 cm of sample, black staining at top 5.08 cm, dry -FILL-	5	10	15	55	10	5				
1	14 11 42 14/5 cm	S2 5	0.91 1.43		125.49 0.91		Note: When driving casing encountered possible 15.24 cm cobble at approximately 0.91 m. Poor recovery for S2. Note: Large cobble or boulder at 1.22 m. Very hard drilling. Encountered multiple cobbles when driving casing to 1.43 m.										
					124.97 1.43		Note: Attempted roller bit and split spoon to 1.43 m. Refusal on a possible granite block or steel plate. Driller lost water return. Hole terminated at 1.43 m (no rock core attempted at 1.43 m due to no water return). BOTTOM OF EXPLORATION 1.43 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

60.96

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram
Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary
Overburden (lin. m) 1.43
Rock Cored (lin. m)
Samples 2S

Boring No. HA-B1 **METRIC**

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High
Toughness: L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH — TEST BORING REPORT

METRIC — Boring No. **HA-B1A**

Project: BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location: BRANDON, VERMONT
Client: CLD CONSULTING ENGINEERS, INC.
Contractor: NEW ENGLAND BORING CONTRACTORS, INC.

File No. 41107-100
Sheet No. 1 of 1
Start 4 Aug 2015
Finish 4 Aug 2015
Driller M. Thompson
H&A Rep. M. Hatton
Elevation 126.40 m (est.)
Datum NAVD 88
Location See Plan

	Casing	Sampler	Barrel
Type	HW	S	--
Inside Diameter (cm)	10.16	3.49	--
Hammer Weight (kg)	136.08	63.50	-
Hammer Fall (cm)	61.0	76.20	-

Drilling Equipment and Procedures
Rig Make & Model: Mobile B57 Track
Bit Type: Roller Bit
Drill Mud: None
Casing: HW Drive to 1.83 m
Hoist/Hammer: Winch, Automatic Hammer

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0			60.96				Note: Second attempt at HA-B1 vicinity. HA-B1A offset approximately 0.71 m southeast of original HA-B1. Note: Advanced casing and roller bit to 1.52 m without sampling.										
1																	
	6 14 30 23	S1 5	1.52 2.13	NO WELL INSTALLED	124.57 1.83	GP	Dense brown poorly graded GRAVEL with sand (GP), mps 3.81 cm, no structure, no odor, dry. Note: Large quartz block or boulder obstructions at 1.83 m. Spoon appears to have bent in between obstructions.	40	35	10	5	5	5				
2							-FILL-										
	13 13 17 35	S2 15	2.13 2.74		123.96 2.44	SP	Medium dense brown poorly graded SAND (SP), mps 2.54 cm, no structure, no odor, dry	5	5	15	40	30	5				
					123.66 2.74		Note: Could not drive casing past 1.83 m. Possible casing crimping due to large obstructions and/or blocks/boulders. Hole terminated at 2.74 m.										
							BOTTOM OF EXPLORATION 2.74 m										
							Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram
Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary
Overburden (lin. m) 2.74
Rock Cored (lin. m)
Samples 2S

Boring No. HA-B1A METRIC

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH — TEST BORING REPORT

METRIC
Boring No. HA-B1B

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	5 Aug 2015
Finish	5 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.40 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	HW	S	NX
Inside Diameter (cm)	10.16	3.49	5.08
Hammer Weight (kg)	136.08	63.50	-
Hammer Fall (cm)	61.0	76.20	-

Drilling Equipment and Procedures
Rig Make & Model: Mobile B57 Track
Bit Type: Roller Bit
Drill Mud: None
Casing: HW Drive to 5.18 m
Hoist/Hammer: Winch, Automatic Hammer

60.96

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED			Note: Third attempt at HA-B1 vicinity. HA-B1B offset approximately 0.71 m north of HA-B1A.										
							Note: Advanced casing and roller bit to 2.74 m without sampling.										
1																	
2																	
3	55 16 9 7	S1 15	2.74 3.35			SP	Medium dense brown poorly graded SAND with gravel (SP), mps 3.18 cm., no structure, no odor, wet (from drilling wash) -FILL-	5	10	10	15	60					
							Note: Refusal at 3.51 m.										
					122.90 3.51		Note: Used core barrel and cored through boulder 3.51 to 3.81 m.										
4					122.59 3.81		Note: Drilling wash at 3.96 m contains possible brick and ceramic fragments										
	3 1 1 4	S2 15	4.27 4.88		122.13 4.27	OL/OH	Very soft dark brown sandy ORGANIC SOILS (OL/OH), mps 3.18 cm., no structure, slight organic odor, wet, brick, glass, ceramics, wood chips, and saw dust throughout sample	5	5	5	5	10	70				
5																	

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram
Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary
Overburden (lin. m) 5.18
Rock Cored (lin. m)
Samples 2S

Boring No. HA-B1B METRIC

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None Plasticity: N-Nonplastic, L-Low, M-Medium, H-High
Toughness: L-Low, M-Medium, H-High Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

TEST BORING REPORT

HALEY ALDRICH

Boring No. HA-B1B
File No. 41107-100
Sheet No. 2 of 1

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
5					121.22 5.18		PROBABLE TOP OF BEDROCK 5.18 m										
					120.98 5.43		Note: Advanced roller bit to 5.43 m. Driller ran out of water. Hole terminated at 5.43 m.										
							BOTTOM OF EXPLORATION 5.43 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

NOTE: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH — TEST BORING REPORT

METRIC
Boring No. HA-B1C

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	6 Aug 2015
Finish	6 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.40 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	HW	-	--
Inside Diameter (cm)	10.16	-	--
Hammer Weight (kg)	136.08	-	-
Hammer Fall (cm)	61.0	-	-

Drilling Equipment and Procedures

- Rig Make & Model: Mobile B57 Track
- Bit Type: Roller Bit
- Drill Mud: None
- Casing: HW Drive to 2.44 m
- Hoist/Hammer: Winch, None

60.96

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
0				NO WELL INSTALLED			Note: 4th attempt at HA-B1 vicinity. HA-B1C offset approximately 0.61 m southwest of HA-B1B. Note: Advanced casing and roller bit to 2.44 m without sampling.										
1																	
2							Note: Casing refusal at 2.44 m on possible obstruction.										
					123.96 2.44		Note: Advanced roller bit to 2.44 m where possible crimping on casing due to obstruction occurred. Hole terminated at 2.44 m due to broken equipment. BOTTOM OF EXPLORATION 2.44 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification

- O Open End Rod
- T Thin Wall Tube
- U Undisturbed Sample
- S Split Spoon
- G Geoprobe

Well Diagram

- Riser Pipe
- Screen
- Filter Sand
- Cuttings
- Grout
- Concrete
- Bentonite Seal

Summary

- Overburden (lin. m) 2.44
- Rock Cored (lin. m)
- Samples -

Boring No. HA-B1C METRIC

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH — TEST BORING REPORT

METRIC
Boring No. HA-B2

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	5 Aug 2015
Finish	5 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.34 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel	Drilling Equipment and Procedures
Type	HW	S	--	Rig Make & Model: Mobile B57 Track
Inside Diameter (cm)	10.16	3.49	--	Bit Type: Roller Bit; Drill Mud: None
Hammer Weight (kg)	136.08	63.50	-	Casing: HW Drive to 2.59 m
Hammer Fall (cm)	61.0	76.20	-	Hoist/Hammer: Winch, Automatic Hammer

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	60.96 Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
0				NO WELL INSTALLED			Note: Used roller bit to cut through asphalt. Note: 15.24 cm asphalt. -ASPHALT-										
	42 12 6 4	S1 25	~~0.15~~ 0.76		126.19 0.15	SM	Medium dense dark brown silty SAND with gravel (SM), mps 1.27 cm, no structure, asphalt-like odor, moist, upper 15.24 cm of sample possibly reclaimed asphalt mixed in with the soil matrix		15	10	20	35	20				
1	5 8 6	S2 25	~~0.76~~ 1.22		125.58 0.76	SP-SM	Medium dense red-brown poorly graded SAND with silt (SP-SM), mps 1.27 cm, no structure, no odor, moist -FILL-		10	5	20	55	10				
	6	S2A	1.22 ~~1.37~~		125.12 1.22	SW	Brown well graded SAND with gravel (SW), mps 3.81 cm, no structure, no odor, moist	25	20	15	15	20	5				
	4 1 1	S3 15	1.52 1.98		124.82 1.52	SM	Very loose dark brown silty SAND (SM), mps 0.64 cm, no structure, no odor, wet (from drilling wash) -FOREST MAT-		5	5	10	55	25				
2	2 1 1 6	S4 30	1.98 2.59		124.36 1.98	OL/OH	Loose dark brown ORGANIC SOIL (OL/OH), mps 1.27 cm, no structure, organic odor, wet PROBABLE TOP OF BEDROCK 2.59 m		5			5	90				
					123.75 2.59		Note: Probable bedrock at 2.59 m, advanced roller bit to 3.05 m, consistent rock chips in composition and color. -PROBABLE BEDROCK-										
					123.29 3.05		BOTTOM OF EXPLORATION 3.05 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
- O Open End Rod
- T Thin Wall Tube
- U Undisturbed Sample
- S Split Spoon
- G Geoprobe

Well Diagram
- Riser Pipe
- Screen
- Filter Sand
- Cuttings
- Grout
- Concrete
- Bentonite Seal

Summary
- Overburden (lin. m) 2.59
- Rock Cored (lin. m)
- Samples 5S

Boring No. HA-B2 METRIC

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH

TEST BORING REPORT

METRIC
Boring No. HA-P1

Project BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location BRANDON, VERMONT
Client CLD CONSULTING ENGINEERS, INC.
Contractor NEW ENGLAND BORING CONTRACTORS, INC.

File No. 41107-100
Sheet No. 1 of 1
Start 5 Aug 2015
Finish 5 Aug 2015
Driller M. Thompson
H&A Rep. K. Russ
Elevation 126.40 m (est.)
Datum NAVD 88
Location See Plan

	Casing	Sampler	Barrel	Drilling Equipment and Procedures
Type	-	-	--	Rig Make & Model: Mobile B57 Track
Inside Diameter (cm)		-	--	Bit Type: Roller Bit; Drill Mud: None
Hammer Weight (kg)	-	-	-	Casing: None
Hammer Fall (cm)	-	-	-	Hoist/Hammer: None, None

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.28		Note: Used roller bit to cut through asphalt.										
		S1	0.12		0.12	SW	Note: 11.43 cm asphalt. -ASPHALT-		47	24	21	6	2				
			0.18		126.19												
					0.21		Dark brown well graded SAND with gravel (SW), mps 1.91 cm, no structure, asphalt-like odor, wet (due to drilling wash) Note: Sample was collected manually. -FILL-										
							BOTTOM OF EXPLORATION 0.21 m										
							Note: Upon completion patched surface with asphalt patch.										

Water Level Data						Sample Identification	Well Diagram	Summary
Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)	O Open End Rod	Riser Pipe	Overburden (lin. m) 0.21
		Not observed				T Thin Wall Tube	Screen	Rock Cored (lin. m)
						U Undisturbed Sample	Filter Sand	Samples 1S
						S Split Spoon	Cuttings	Boring No. HA-P1
						G Geoprobe	Grout	METRIC
							Concrete	
							Bentonite Seal	

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None Toughness: L-Low, M-Medium, H-High Plasticity: N-Nonplastic, L-Low, M-Medium, H-High Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HA-LIB09-BOS.GLB H&A TEST BORING METRIC CONVERT(CM)-09 \\HALEYALDRICH.COM\SHARE\MAN_COMMON\41107_BRANDON VT BRIDGE\100\FIELD\GINT\2015-0812-HAI-TEST BORING HA P1-HA P10 -METRIC.GPJ 9 Jun 17

TEST BORING REPORT

HALEY ALDRICH

METRIC
Boring No. HA-P10 ALT

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	5 Aug 2015
Finish	5 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.31 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	-	-	NX
Inside Diameter (cm)		-	5.08
Hammer Weight (kg)	-	-	-
Hammer Fall (cm)	-	-	-

Drilling Equipment and Procedures

- Rig Make & Model: Mobile B57 Track
- Bit Type: Roller Bit
- Drill Mud: None
- Casing: None
- Hoist/Hammer: None, None

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.22 0.09		Note: Used roller bit to cut through asphalt. Note: 10.16 cm asphalt. -ASPHALT-										
					126.03 0.27		-CONCRETE SLAB- Note: Cored and recovered 17.78 of concrete slab. Reinforcement observed.										
		S1	0.27 0.30		126.00 0.30	GP	Dark brown poorly graded GRAVEL with sand (GP), mps 1.91 cm, light concrete-like odor (probably from coring), wet (from coring) -FILL- Note: Sample was collected manually.		72	16	7	3	2				

BOTTOM OF EXPLORATION 0.30 m

Note: Upon completion patched surface with asphalt patch.

Note: Offset 0.09 m east of HA-P10 ALT to obtain a second concrete core that is intact and suitable for compressive strength laboratory testing.

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification

- O Open End Rod
- T Thin Wall Tube
- U Undisturbed Sample
- S Split Spoon
- G Geoprobe

Well Diagram

- Riser Pipe
- Screen
- Filter Sand
- Cuttings
- Grout
- Concrete
- Bentonite Seal

Summary

- Overburden (lin. m) 0.30
- Rock Cored (lin. m)
- Samples 1S

Boring No. HA-P10 ALT
METRIC

Field Tests:
Dilatancy: R-Rapid, S-Slow, N-None
Toughness: L-Low, M-Medium, H-High
Plasticity: N-Nonplastic, L-Low, M-Medium, H-High
Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH

TEST BORING REPORT

METRIC
Boring No. HA-P2

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	5 Aug 2015
Finish	5 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.40 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	-	-	--
Inside Diameter (cm)		-	--
Hammer Weight (kg)	-	-	-
Hammer Fall (cm)	-	-	-

Drilling Equipment and Procedures

Rig Make & Model: Mobile B57 Track
Bit Type: Roller Bit
Drill Mud: None
Casing: None
Hoist/Hammer: None, None

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.31 0.09		Note: Used roller bit to cut through asphalt. Note: 10.16 cm asphalt. -ASPHALT-										
		S1	0.09 0.34		126.16 0.24	SP-SM	Dark brown poorly graded SAND with gravel and silt (SP-SM), mps 3.18 cm., as trace coarse gravel, no structure, light asphalt-like odor, dry -FILL- Note: Fragments of asphalt within soil sample, probably from existing pavement. Note: Sample was collected manually.		28	22	32	9	9				
							BOTTOM OF EXPLORATION 0.24 m										
							Note: Upon completion patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram
Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary
Overburden (lin. m) 0.24
Rock Cored (lin. m)
Samples 1S

Boring No. HA-P2 METRIC

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High
Toughness: L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

TEST BORING REPORT

METRIC
Boring No. HA-P3

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	5 Aug 2015
Finish	5 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.19 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel	Drilling Equipment and Procedures
Type	-	-	--	Rig Make & Model: None
Inside Diameter (cm)		-	--	Bit Type: None Drill Mud: None
Hammer Weight (kg)	-	-	-	Casing: None
Hammer Fall (cm)	-	-	-	Hoist/Hammer: None, None

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
0				NO WELL INSTALLED	126.03 0.15		Note: Driller used manual tools (no drilling rig) to cut through asphalt. Note: 13.97 cm asphalt. -ASPHALT- Note: Probable concrete slab below asphalt based on drillers observations. Note: No samples collected. BOTTOM OF EXPLORATION 0.15 m Note: Upon completion patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification

O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram

Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary

Overburden (lin. m) 0.15
Rock Cored (lin. m)
Samples -

Boring No. HA-P3 **METRIC**

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH — TEST BORING REPORT

METRIC
Boring No. HA-P4

Project: BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location: BRANDON, VERMONT
Client: CLD CONSULTING ENGINEERS, INC.
Contractor: NEW ENGLAND BORING CONTRACTORS, INC.

File No. 41107-100
Sheet No. 1 of 1
Start 3 Aug 2015
Finish 3 Aug 2015
Driller M. Thompson
H&A Rep. M. Hatton
Elevation 126.49 m (est.)
Datum NAVD 88
Location See Plan

	Casing	Sampler	Barrel	Drilling Equipment and Procedures
Type	HSA	S	NX	Rig Make & Model: Mobile B57 Track
Inside Diameter (cm)	10.16	3.49	5.08	Bit Type: Cutting Head; Drill Mud: None
Hammer Weight (kg)	-	63.50	-	Casing: HSA Spun to 0.12 m
Hammer Fall (cm)	-	76.20	-	Hoist/Hammer: Winch, Automatic Hammer

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.37 / 0.12		Note: Used hollow stem augers with cutting head to cut through asphalt. Note: 12.7 cm asphalt. -ASPHALT-										
	24 10 6 10	S1 25	0.30 0.91		126.22 / 0.27	GW-GM	Note: Cored 12.7 cm and recovered 7.62 cm of concrete slab. Core barrel jammed at 12.7 cm, 5.08 cm of recovery at bottom of hole. Cored an additional 2.54 cm and recovered 7.62 cm (including the 5.08 cm from first core attempt) -CONCRETE SLAB-	23	26	10	15	16	10				
1	10 30 R	S2 10	0.91 1.22		125.27 / 1.22	GW-GM	Note: Core sample includes steel reinforcement at approximate depths of 0 cm and 6.35 cm Punched through slab at 0.27 m, gravel at bottom of hole. Medium dense olive-gray to olive-brown well graded GRAVEL with sand and silt (GW-GM), mps 5.08 cm, no structure, no odor, dry	23	26	10	15	16	10				
							Dense olive-gray well graded GRAVEL with sand and silt (GW-GM), mps 5.08 cm, no structure, no odor, dry										
							BOTTOM OF EXPLORATION 1.22 m										
							Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram
Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary
Overburden (lin. m) 1.22
Rock Cored (lin. m)
Samples 2S

Boring No. HA-P4 **METRIC**

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

TEST BORING REPORT

HALEY ALDRICH

METRIC
Boring No. HA-P5

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	3 Aug 2015
Finish	3 Aug 2015
Driller	M. Thompson
H&A Rep.	M. Hatton
Elevation	126.61 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	HSA	S	NX
Inside Diameter (cm)	10.16	3.49	5.08
Hammer Weight (kg)	-	63.50	-
Hammer Fall (cm)	-	76.20	-

Drilling Equipment and Procedures

Rig Make & Model: Mobile B57 Track
Bit Type: Cutting Head
Drill Mud: None
Casing: HSA Spun to 0.09 m
Hoist/Hammer: Winch,

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
0				NO WELL INSTALLED	126.52 0.09		Note: Used hollow stem auger with cutting head to cut through asphalt. Note: 10.16 cm asphalt. -ASPHALT-										
	21 23 18 12	S1 36	0.24 0.85		126.37 0.24	SM	-CONCRETE SLAB- Note: Cored and recovered 15.24 cm of concrete slab. No reinforcement observed. Dense olive-gray silty SAND with gravel (SM), mps 2.54 cm, no structure, no odor, dry -FILL-	16	27	14	18	9	16				
1	7 R	S2	0.85 1.01		125.61 1.01		No recovery on S2 BOTTOM OF EXPLORATION 1.01 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification

O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram

Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary

Overburden (lin. m) 1.01
Rock Cored (lin. m)
Samples 2S

Boring No. HA-P5 **METRIC**

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None
Toughness: L-Low, M-Medium, H-High
Plasticity: N-Nonplastic, L-Low, M-Medium, H-High
Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH

TEST BORING REPORT

METRIC
Boring No. HA-P6

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	3 Aug 2015
Finish	3 Aug 2015
Driller	M. Thompson
H&A Rep.	M. Hatton
Elevation	126.40 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel	Drilling Equipment and Procedures
Type	HSA	S	NX	Rig Make & Model: Mobile B57 Track
Inside Diameter (cm)	10.16	3.49	5.08	Bit Type: Cutting Head; Drill Mud: None
Hammer Weight (kg)	-	63.50	-	Casing: HSA Spun to 0.12 m
Hammer Fall (cm)	-	76.20	-	Hoist/Hammer: Winch, Automatic Hammer

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.28 0.12		Note: Used hollow stem auger with cutting head to cut through asphalt. Note: 12.7 cm asphalt. -ASPHALT-										
					126.13 0.27		-CONCRETE SLAB- Note: Cored and recovered 15.24 cm of concrete slab. No reinforcement observed.										
	9 12 R	S1 15	0.38 0.69		125.71 0.69	GW-GM	Note: Cobbles at bottom of core barrel (approximately 5.72 cm in size) punched through into sand. Medium dense brown well graded GRAVEL with sand and silt (GW-GM), mps 5.08 cm, no structure, no odor, possible cobble at bottom of spoon -FILL-	29	31	9	9	12	10				
							BOTTOM OF EXPLORATION 0.70 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification

- O Open End Rod
- T Thin Wall Tube
- U Undisturbed Sample
- S Split Spoon
- G Geoprobe

Well Diagram

- Riser Pipe
- Screen
- Filter Sand
- Cuttings
- Grout
- Concrete
- Bentonite Seal

Summary

Overburden (lin. m) 0.70
Rock Cored (lin. m)
Samples 1S

Boring No. HA-P6 **METRIC**

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH

TEST BORING REPORT

METRIC
Boring No. HA-P7

Project BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location BRANDON, VERMONT
Client CLD CONSULTING ENGINEERS, INC.
Contractor NEW ENGLAND BORING CONTRACTORS, INC.

File No. 41107-100
Sheet No. 1 of 1
Start 3 Aug 2015
Finish 3 Aug 2015
Driller M. Thompson
H&A Rep. M. Hatton
Elevation 126.49 m (est.)
Datum NAVD 88
Location See Plan

	Casing	Sampler	Barrel	Drilling Equipment and Procedures
Type	HSA	S	NX	Rig Make & Model: Mobile B57 Track
Inside Diameter (cm)	10.16	3.49	5.08	Bit Type: Cutting Head; Drill Mud: None
Hammer Weight (kg)	-	63.50	-	Casing: HSA Spun to 0.09 m
Hammer Fall (cm)	-	76.20	-	Hoist/Hammer: Winch, Automatic Hammer

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Dilatancy	Toughness	Plasticity	Strength
0				NO WELL INSTALLED	126.40 0.09		Note: Used hollow stem auger with cutting head to cut through asphalt. Note: 10.16 cm asphalt. -ASPHALT-										
	9 17 10 12	S1 30	0.24 0.85		126.25 0.24	SM	-CONCRETE SLAB- Note: Cored and recovered 15.24 cm of concrete slab. No reinforcement observed.	11	31	14	17	12	15				
	R	S2	0.85 0.85		125.64 0.85		Medium dense olive-gray to olive-brown silty SAND with gravel (SM), mps 5.08 cm, no structure no odor, dry Immediate refusal on S2 -FILL-										
							BOTTOM OF EXPLORATION 0.85 m Note: Upon completion grouted hole and patched surface with asphalt patch.										

Water Level Data					
Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification
- O Open End Rod
- T Thin Wall Tube
- U Undisturbed Sample
- S Split Spoon
- G Geoprobe

Well Diagram
- Riser Pipe
- Screen
- Filter Sand
- Cuttings
- Grout
- Concrete
- Bentonite Seal

Summary
Overburden (lin. m) 0.85
Rock Cored (lin. m)
Samples 2S

Boring No. HA-P7 **METRIC**

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None; Toughness: L-Low, M-Medium, H-High; Plasticity: N-Nonplastic, L-Low, M-Medium, H-High; Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

HALEY ALDRICH

TEST BORING REPORT

METRIC
Boring No. HA-P9

Project	BRIDGE NO. 114 US ROUTE 7 OVER NESHOBE RIVER
Location	BRANDON, VERMONT
Client	CLD CONSULTING ENGINEERS, INC.
Contractor	NEW ENGLAND BORING CONTRACTORS, INC.

File No.	41107-100
Sheet No.	1 of 1
Start	5 Aug 2015
Finish	5 Aug 2015
Driller	M. Thompson
H&A Rep.	K. Russ
Elevation	126.55 m (est.)
Datum	NAVD 88
Location	See Plan

	Casing	Sampler	Barrel
Type	-	-	NX
Inside Diameter (cm)		-	5.08
Hammer Weight (kg)	-	-	-
Hammer Fall (cm)	-	-	-

Drilling Equipment and Procedures

Rig Make & Model: Mobile B57 Track
Bit Type: Roller Bit
Drill Mud: None
Casing: None
Hoist/Hammer: None, None

Depth (m)	Sampler Blows per 15 cm	Sample No. & Rec. (cm)	Sample Depth (m)	Well Diagram	Elev./Depth (m)	USCS Symbol	Visual-Manual Identification and Description (Density/consistency, color, GROUP NAME, structure, odor, moisture, optional descriptions, geologic interpretation)	Gravel % Coarse	Gravel % Fine	Sand % Coarse	Sand % Medium	Sand % Fine	% Fines	Field Test Dilatancy	Field Test Toughness	Field Test Plasticity	Field Test Strength
0				NO WELL INSTALLED	126.46 0.09		Note: Used roller bit to cut through asphalt. Note: 8.89 cm asphalt. -ASPHALT-										
					126.28 0.27		-CONCRETE SLAB- Note: Cored and recovered 19.05 cm of concrete slab. Reinforcement observed.										
		S1	0.27 0.37		126.19 0.37	GP-GM	Dark brown poorly graded GRAVEL with silt and sand (GP-GM), mps 2.54 cm, as trace coarse gravel, wet (due to coring), concrete-like odor (probably due to coring) -FILL- Note: Sample was collected manually.		48	15	15	10	12				
							BOTTOM OF EXPLORATION 0.37 m Note: Upon completion patched surface with asphalt patch.										

Water Level Data

Date	Time	Elapsed Time (hr.)	Depth (m) to: Bottom of Casing	Bottom of Hole	Water (+/-)
		Not observed			

Sample Identification

O Open End Rod
T Thin Wall Tube
U Undisturbed Sample
S Split Spoon
G Geoprobe

Well Diagram

Riser Pipe
Screen
Filter Sand
Cuttings
Grout
Concrete
Bentonite Seal

Summary

Overburden (lin. m) 0.37
Rock Cored (lin. m)
Samples 1S

Boring No. HA-P9 METRIC

Field Tests: Dilatancy: R-Rapid, S-Slow, N-None — Plasticity: N-Nonplastic, L-Low, M-Medium, H-High
Toughness: L-Low, M-Medium, H-High — Dry Strength: N-None, L-Low, M-Medium, H-High, V-Very High

Note: Soil identification based on visual-manual methods of the USCS as practiced by Haley & Aldrich, Inc.

APPENDIX B

Geotechnical Laboratory Test Results

GeoTesting
EXPRESS

Technologies to manage risk for infrastructure

Boston
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New York
www.geotesting.com

Transmittal

TO:

Meghan Hatton
Haley & Aldrich, Inc.
3 Bedford Farms Drive
Bedford, NH 03110

DATE: 8/27/2015	GTX NO: 303588
RE: Bridge No. 114, US Route 7 over the Neshobe River	

COPIES	DATE	DESCRIPTION
	8/27/2015	**August 2015 Laboratory Test Report**

REMARKS:

CC:

SIGNED:

Joe Tomei, Laboratory Manager

APPROVED BY:

Mark Dobday, P.G., Laboratory Manager

GeoTesting
EXPRESS

Technologies to manage risk for infrastructure

Boston
Atlanta
Chicago
Los Angeles
New York
www.geotesting.com

August 27, 2015

Meghan Hatton
Haley & Aldrich, Inc.
3 Bedford Farms Drive
Bedford, NH 03110

RE: Bridge No. 114, US Route 7 over the Neshobe River, Brandon, VT (GTX-303588)

Dear Meghan:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received eight samples from you on 8/13/2015. These samples were labeled as follows:

Boring	Sample	Depth
HA-P1	S1	0.4-0.7 ft
HA-P2	S1	0.3-0.8 ft
HA-P4	S1 and S2	1.0-4.0 ft
HA-P5	S1	0.8-2.8 ft
HA-P6	S1	1.3-2.3 ft
HA-P7	S1	0.8-2.8 ft
HA-P9	S1	0.9-1.2 ft
HA-P10 Alt.	S1	0.9-1.0 ft

GTX performed the following test on each of these samples:

ASTM D422 - Grain Size Analysis - Sieve Only

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

GeoTesting
EXPRESS

Technologies to manage risk for infrastructure

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Geotechnical Test Report **8/27/2015**

GTX-303588
Bridge No. 114, US Route 7 over the Neshobe River

Brandon, VT

Client Project No.: 41107-100

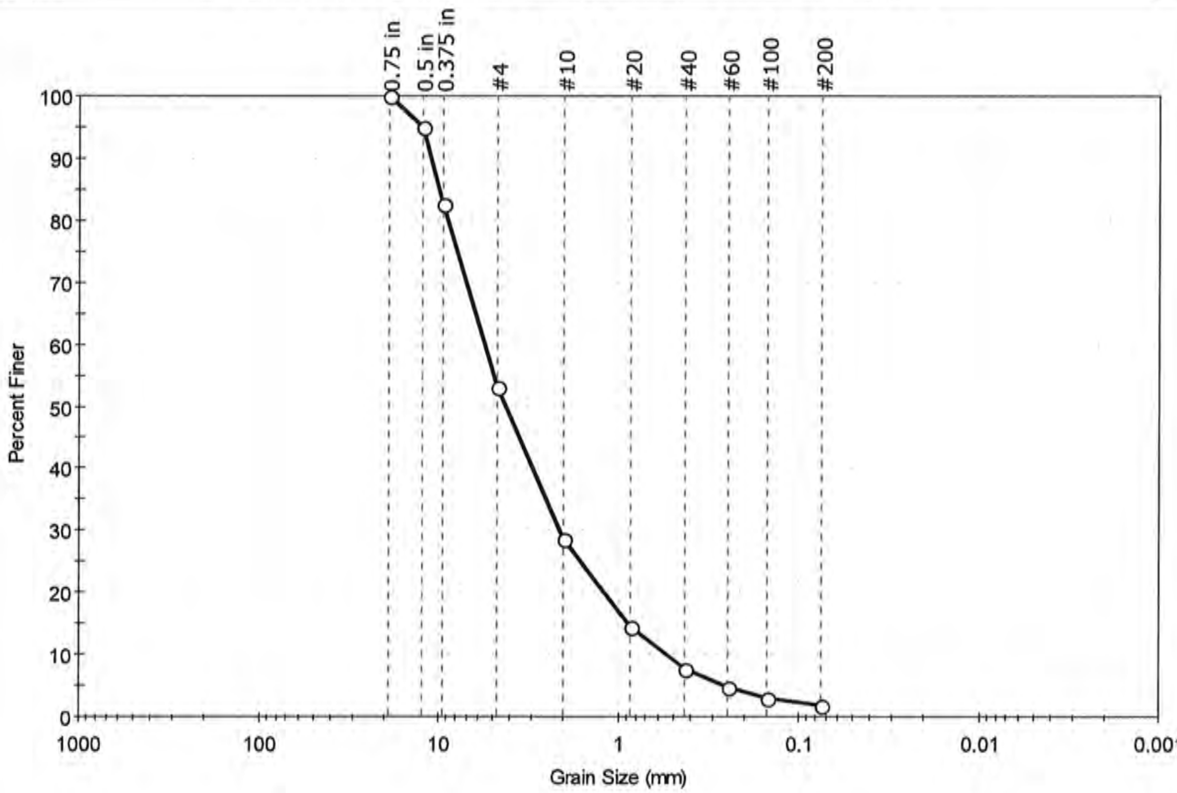
Prepared for:

Haley & Aldrich, Inc.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.				
Project:	Bridge No. 114, US Route 7 over the Neshobe River				
Location:	Brandon, VT			Project No:	GTX-303588
Boring ID:	HA-P1	Sample Type:	bag	Tested By:	jbr
Sample ID:	S1	Test Date:	08/20/15	Checked By:	emm
Depth :	0.4-0.7 ft	Test Id:	343813		
Test Comment:	---				
Visual Description:	Moist, dark gray sand with gravel				
Sample Comment:	--				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	47.0	51.2	1.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	83		
#4	4.75	53		
#10	2.00	29		
#20	0.85	14		
#40	0.42	8		
#60	0.25	5		
#100	0.15	3		
#200	0.075	1.8		

Coefficients

D_{85} = 10.0381 mm D_{30} = 2.1022 mm

D_{60} = 5.6030 mm D_{15} = 0.8815 mm

D_{50} = 4.2771 mm D_{10} = 0.5366 mm

C_u = 10.442 C_c = 1.470

Classification

ASTM Well-graded sand with gravel (SW)

AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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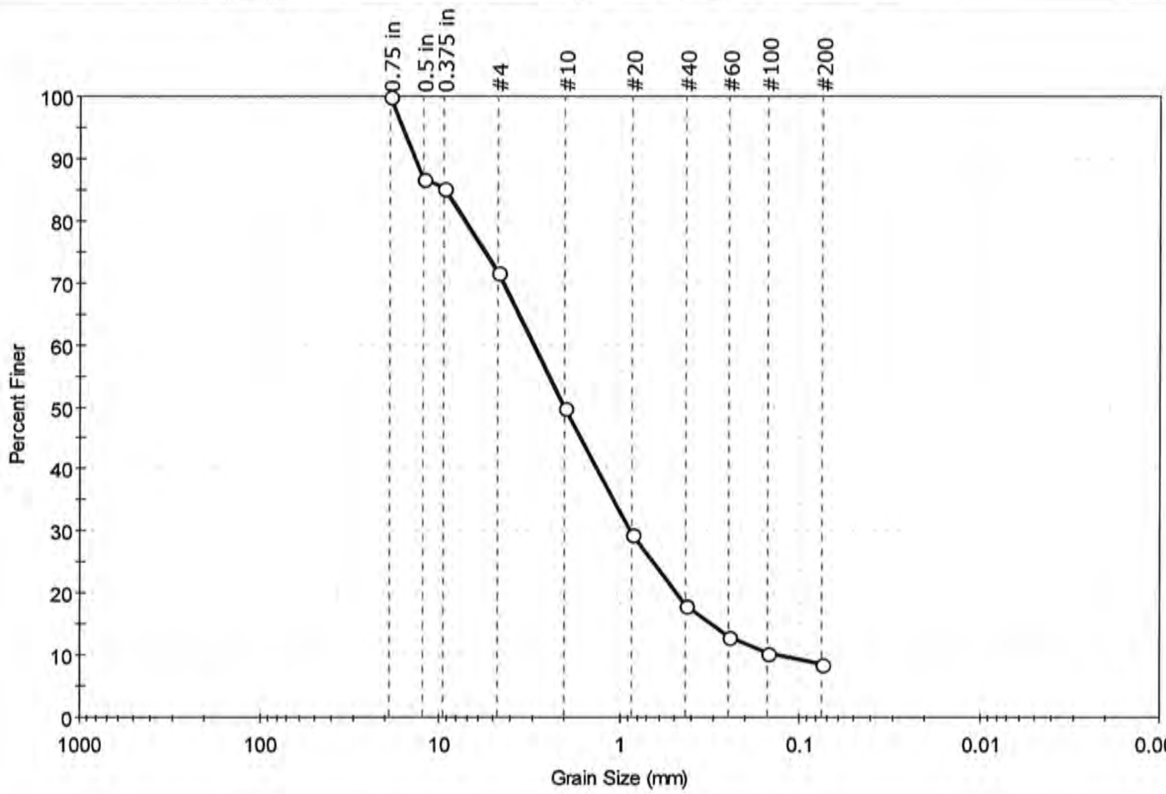
Client:	Haley & Aldrich, Inc.		
Project:	Bridge No. 114, US Route 7 over the Neshobe River		
Location:	Brandon, VT	Project No:	GTX-303588
Boring ID: HA-P2	Sample Type: bag	Tested By:	jbr
Sample ID: S1	Test Date: 08/20/15	Checked By:	emm
Depth : 0.3-0.8 ft	Test Id: 343810		
Test Comment:	---		
Visual Description:	Moist, black sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	28.4	63.1	8.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	87		
0.375 in	9.50	85		
#4	4.75	72		
#10	2.00	50		
#20	0.85	29		
#40	0.42	18		
#60	0.25	13		
#100	0.15	10		
#200	0.075	8.5		

Coefficients

D_{85} =9.3231 mm	D_{30} =0.8710 mm
D_{60} =2.9998 mm	D_{15} =0.3094 mm
D_{50} =2.0197 mm	D_{10} =0.1321 mm
C_u =22.709	C_c =1.914

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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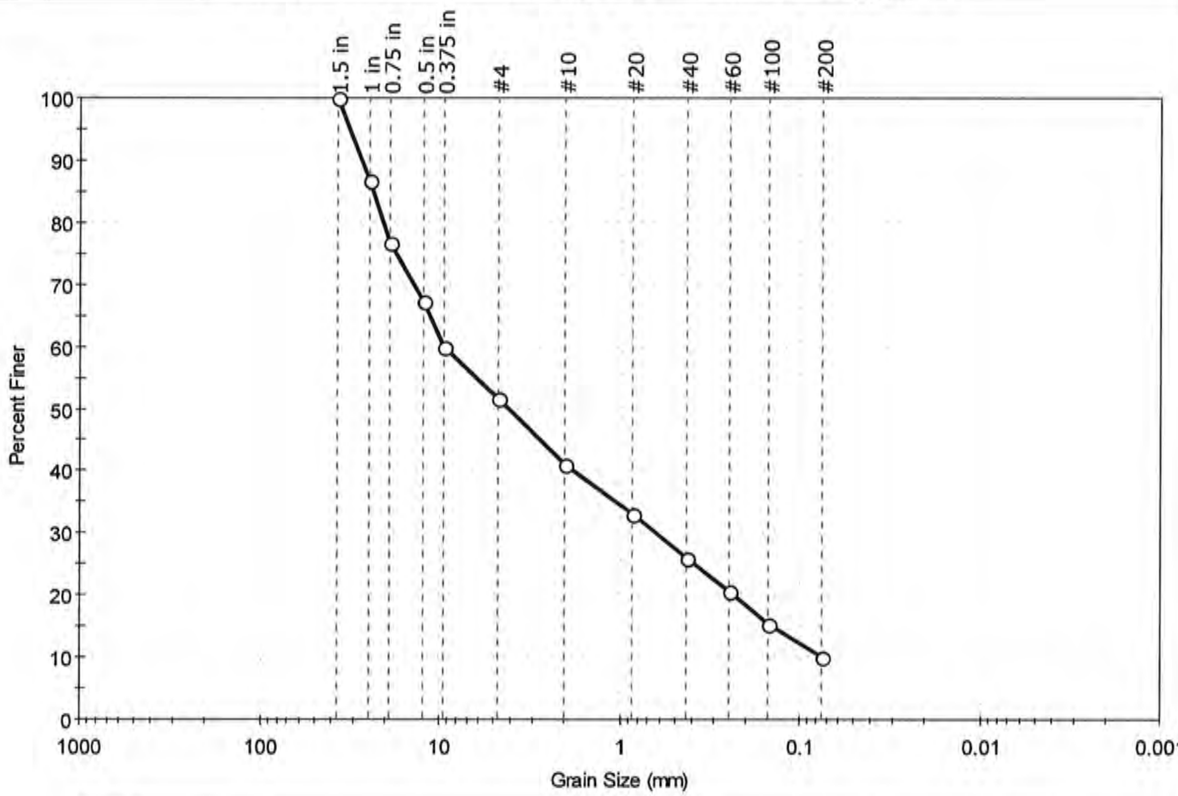
Client:	Haley & Aldrich, Inc.		
Project:	Bridge No. 114, US Route 7 over the Neshobe River		
Location:	Brandon, VT	Project No:	GTX-303588
Boring ID: HA-P4	Sample Type: bag	Tested By:	jbr
Sample ID: S1 and S2	Test Date: 08/20/15	Checked By:	emm
Depth : 1.0-4.0 ft	Test Id: 343807		
Test Comment:	---		
Visual Description:	Moist, dark yellowish brown gravel with silt and sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	48.5	41.5	10.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	87		
0.75 in	19.00	77		
0.5 in	12.50	67		
0.375 in	9.50	60		
#4	4.75	51		
#10	2.00	41		
#20	0.85	33		
#40	0.42	26		
#60	0.25	21		
#100	0.15	15		
#200	0.075	10		

Coefficients

D_{85} =23.7908 mm — D_{30} =0.6299 mm

D_{60} =9.5194 mm — D_{15} =0.1435 mm

D_{50} =4.2029 mm — D_{10} =N/A

C_u =N/A — C_c =N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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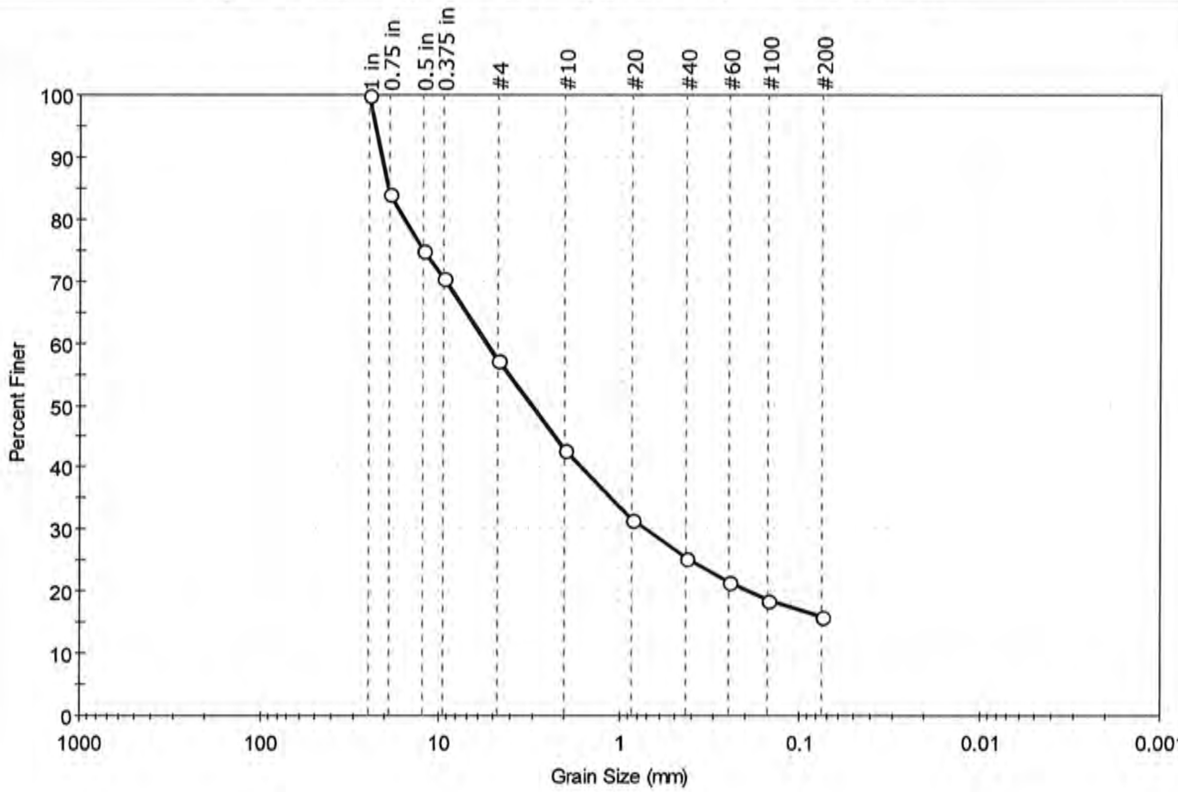
Client: Haley & Aldrich, Inc.			
Project: Bridge No. 114, US Route 7 over the Neshobe River			
Location: Brandon, VT		Project No:	GTX-303588
Boring ID: HA-P5	Sample Type: bag	Tested By:	jbr
Sample ID: S1	Test Date: 08/20/15	Checked By:	emm
Depth : 0.8-2.8 ft	Test Id: 343811		
Test Comment: ---			
Visual Description: Moist, grayish brown silty gravel with sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	42.8	41.3	15.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	84		
0.5 in	12.50	75		
0.375 in	9.50	71		
#4	4.75	57		
#10	2.00	43		
#20	0.85	32		
#40	0.42	25		
#60	0.25	22		
#100	0.15	19		
#200	0.075	16		

Coefficients

D_{85} =19.3058 mm D_{30} =0.7111 mm
D_{60} =5.4960 mm D_{15} =N/A
D_{50} =3.0822 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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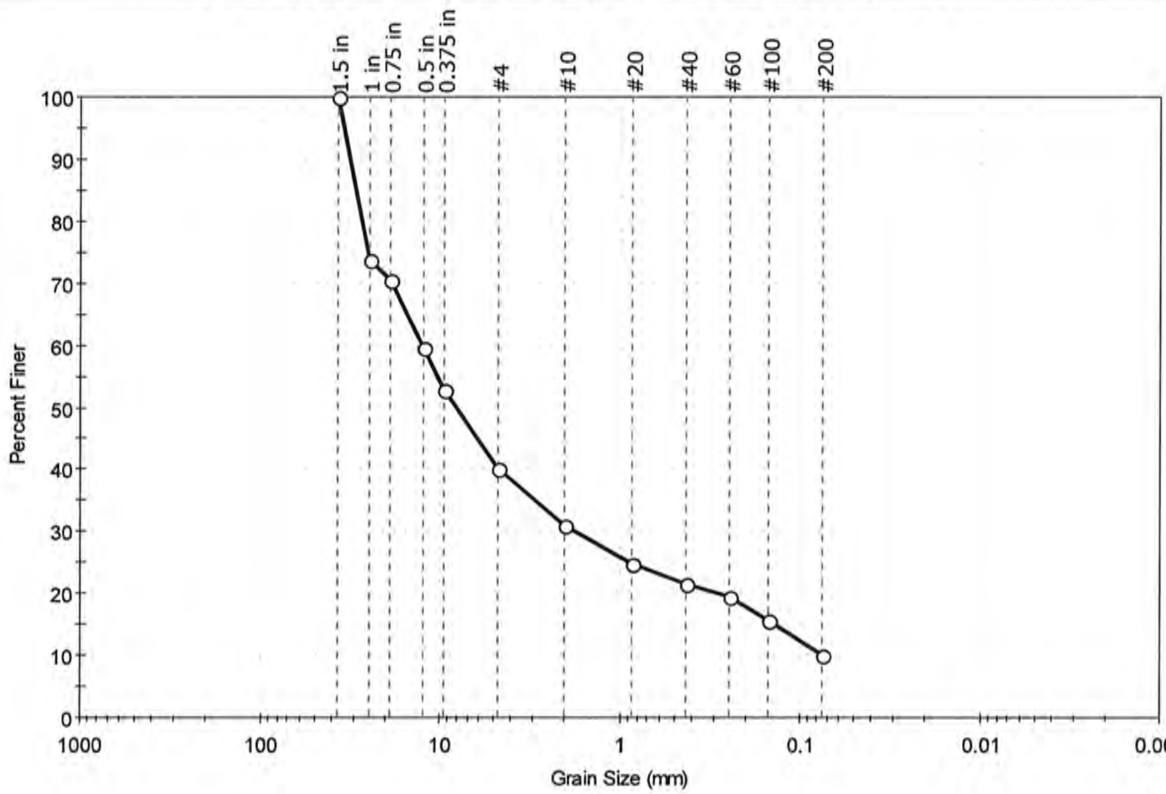
Client:	Haley & Aldrich, Inc.		
Project:	Bridge No. 114, US Route 7 over the Neshobe River		
Location:	Brandon, VT	Project No:	GTX-303588
Boring ID: HA-P6	Sample Type: bag	Tested By:	jbr
Sample ID: S1	Test Date: 08/20/15	Checked By:	emm
Depth : 1.3-2.3 ft	Test Id: 343812		

Test Comment: ---
Visual Description: Moist, pale yellow gravel with silt and sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	59.9	30.0	10.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	74		
0.75 in	19.00	71		
0.5 in	12.50	60		
0.375 in	9.50	53		
#4	4.75	40		
#10	2.00	31		
#20	0.85	25		
#40	0.42	22		
#60	0.25	20		
#100	0.15	15		
#200	0.075	10		

Coefficients

D_{85} = 29.7690 mm D_{30} = 1.7713 mm
D_{60} = 12.6785 mm D_{15} = 0.1409 mm
D_{50} = 8.1391 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD

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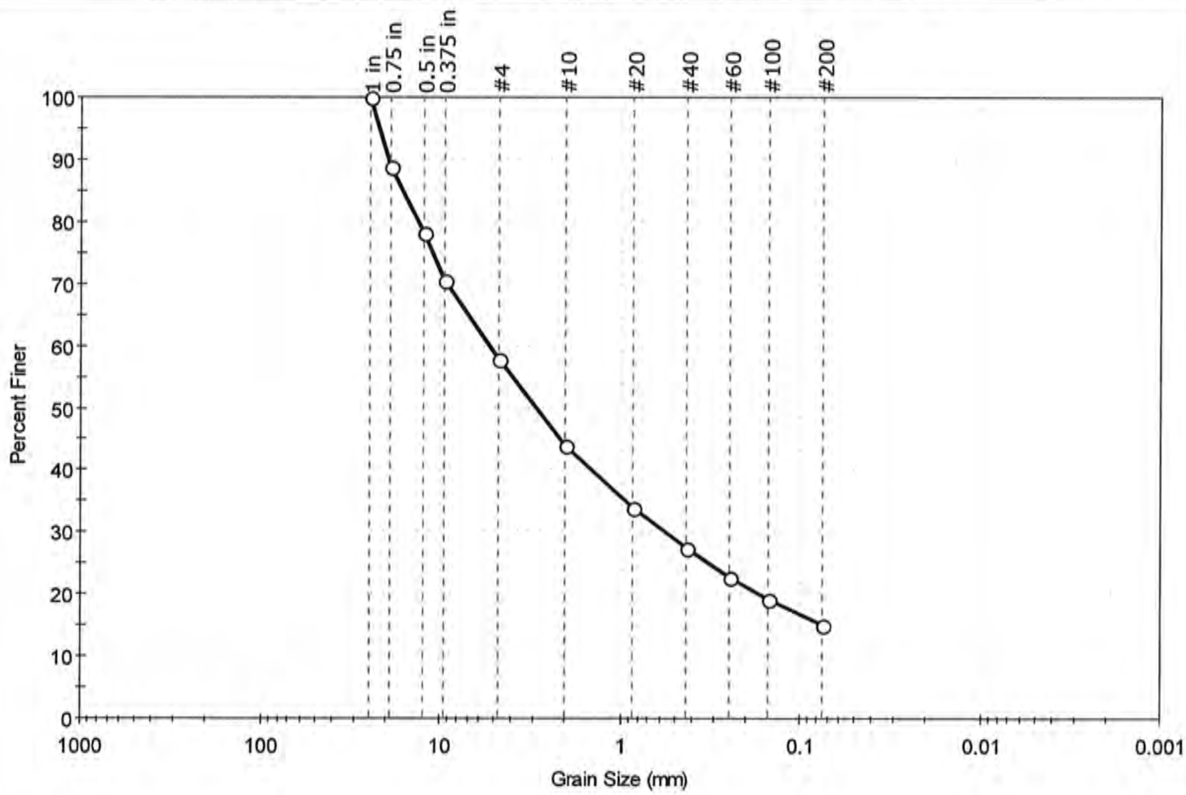
Client:	Haley & Aldrich, Inc.		
Project:	Bridge No. 114, US Route 7 over the Neshobe River		
Location:	Brandon, VT	Project No:	GTX-303588
Boring ID: HA-P7	Sample Type: bag	Tested By:	jbr
Sample ID: S1	Test Date: 08/20/15	Checked By:	emm
Depth : 0.8-2.8 ft	Test Id: 343808		
Test Comment:	---		
Visual Description:	Moist, very dark brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	42.2	42.7	15.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	89		
0.5 in	12.50	78		
0.375 in	9.50	70		
#4	4.75	58		
#10	2.00	44		
#20	0.85	34		
#40	0.42	27		
#60	0.25	23		
#100	0.15	19		
#200	0.075	15		

Coefficients

D_{85} =16.3091 mm D_{30} =0.5578 mm
D_{60} =5.3710 mm D_{15} =N/A
D_{50} =2.9324 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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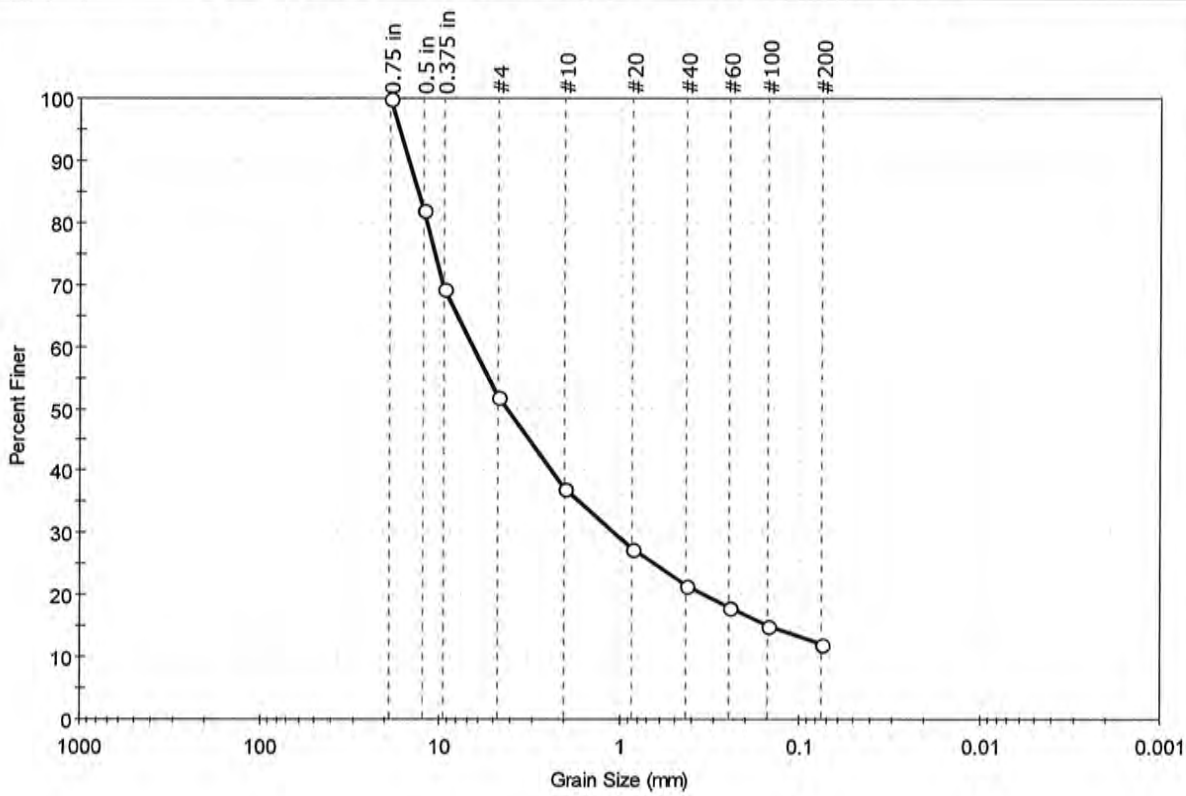
Client:	Haley & Aldrich, Inc.				
Project:	Bridge No. 114, US Route 7 over the Neshobe River				
Location:	Brandon, VT			Project No:	GTX-303588
Boring ID:	HA-P9	Sample Type:	bag	Tested By:	jbr
Sample ID:	S1	Test Date:	08/20/15	Checked By:	emm
Depth :	0.9-1.2 ft	Test Id:	343814		
Test Comment:	---				
Visual Description:	Moist, very dark brown silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	48.1	39.7	12.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	82		
0.375 in	9.50	69		
#4	4.75	52		
#10	2.00	37		
#20	0.85	27		
#40	0.42	22		
#60	0.25	18		
#100	0.15	15		
#200	0.075	12		

Coefficients

D_{85} = 13.3782 mm — D_{30} = 1.0675 mm

D_{60} = 6.5389 mm — D_{15} = 0.1452 mm

D_{50} = 4.2468 mm — D_{10} = N/A

C_u = N/A — C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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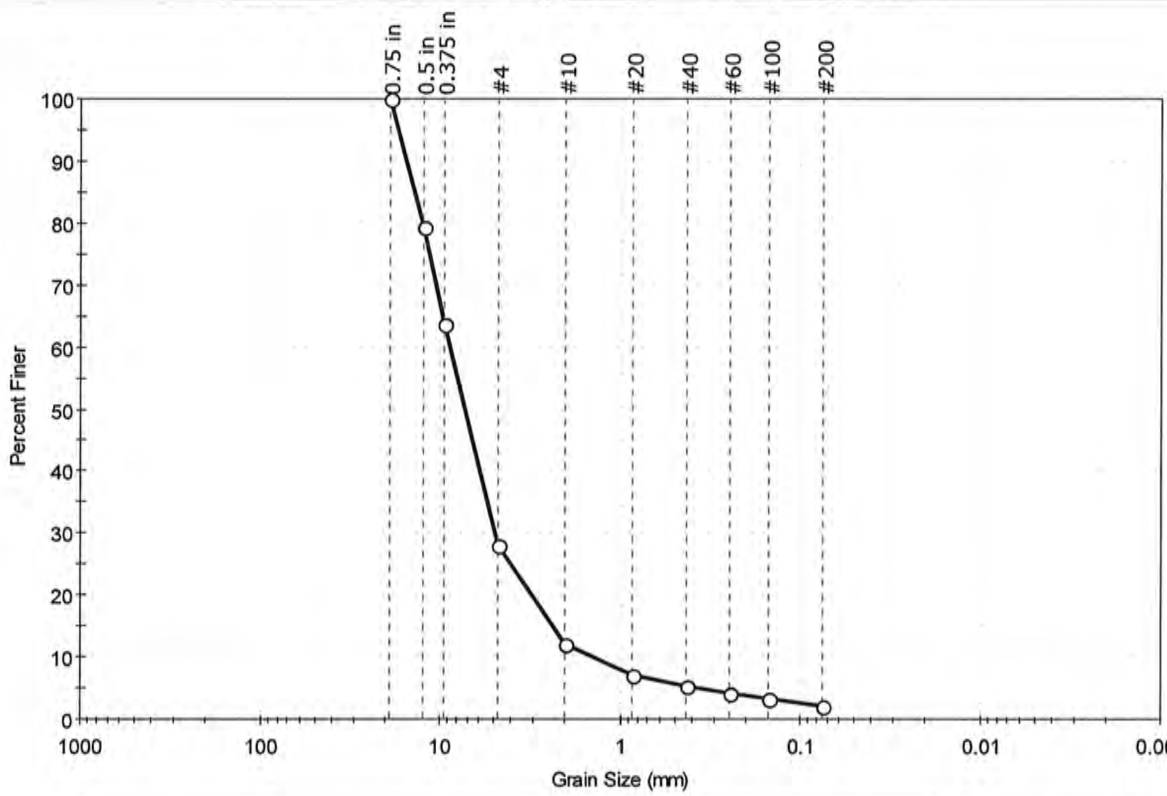
GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.				
Project:	Bridge No. 114, US Route 7 over the Neshobe River				
Location:	Brandon, VT			Project No:	GTX-303588
Boring ID:	HA-P10 Alt.	Sample Type:	bag	Tested By:	jbr
Sample ID:	S1	Test Date:	08/20/15	Checked By:	emm
Depth :	0.9-1.0 ft	Test Id:	343809		
Test Comment:	---				
Visual Description:	Moist, dark grayish brown gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	72.0	25.9	2.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	79		
0.375 in	9.50	64		
#4	4.75	28		
#10	2.00	12		
#20	0.85	7		
#40	0.42	5		
#60	0.25	4		
#100	0.15	3		
#200	0.075	2.1		

Coefficients

D_{85} = 14.0115 mm D_{30} = 4.9357 mm
D_{60} = 8.8426 mm D_{15} = 2.3473 mm
D_{50} = 7.2807 mm D_{10} = 1.4014 mm
C_u = 6.310 C_c = 1.966

Classification

ASTM   Well-graded gravel with sand (GW)

AASHTO   Stone Fragments, Gravel and Sand (A-1-a (1))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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GeoTesting
EXPRESS

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Acton, MA 01720
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2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT	
Company: Haley & Aldrich	
Address: 3 Bedford Farms Drive	
City, State, Zip: Bedford, NH 03110	
Contact: Meghan Hatton	Phone: 603 391 3326
E-mail: mhatton@haleyaldrich.com	Cell: 857 383 8603

INVOICE (complete if different from Client)	
Company:	
Address:	
City, State, Zip:	
Contact:	Phone:
E-mail:	Cell:

PROJECT		
Project Name: Bridge No. 114, US Route 7 over the Neshobe River	Client Project #: 41107-100	Purchase Order#:
Project Location: Brandon, VT	GTX Sales Order #:	Requested Turnaround:
On-site Contact:	E-mail:	Phone:

Boring ID	Sample ID	Depth	Atterberg Limits (ASTM D 4318)	USCS – Classification (ASTM D 2487)	Grain Size: ASTM D 422 ☑ Sieve Only ☑ Sieve & Hydrometer ☐	Density: ASTM D 2937 ☐ ASTM D 7263 ☐	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	pH (ASTM D 4972)	Specific Gravity (ASTM D 854)	Electrical Resistivity (ASTM G 57)	Proctor Compaction: Standard – ASTM D 698 ☐ Modified – ASTM D 1557 ☐	California Bearing Ratio * (ASTM D 1883)	Direct Shear* (ASTM D 3080)	Triaxial Shear* : UU – ASTM D 2850 ☐ CU – ASTM D 4767 ☐ CD – ASTM D7181 ☐	Incremental Consolidation* (ASTM D 2435)	Permeability/ Hydraulic Conductivity* : Fixed Wall – ASTM D 2434 ☐ Flexible Wall – ASTM D 5084 ☐	Unconfined Compression (ASTM D 2166)	Other:	Other:
HA-P4	S1 and S2	1.0-4.0 ft			x															
HA-P7	S1	0.8-2.8 ft			x															
HA-P10 Alt.	S1	0.9-1.0 ft			x															
HA-P2	S1	0.3-0.8 ft			x															
HA-P5	S1	0.8-2.8 ft			x															
HA-P6	S1	1.3-2.3 ft			x															
HA-P1	S1	0.4-0.7 ft			x															
HA-P9	S1	0.9-1.2 ft			x															

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: Meghan Hatton PRINT NAME: Meghan Hatton DATE: [illegible]/12/2015

For GTX Use Only
Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By:	DATE: TIME:	Received By:	DATE: TIME:
Relinquished By:	DATE: TIME:	Received By:	DATE: TIME:

GeoTesting
EXPRESS

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
PI	plasticity index
i	gradient
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3)/2$, $(\sigma_v + \sigma_h)/2$
p'	$(\sigma'_1 + \sigma'_3)/2$, $(\sigma'_v + \sigma'_h)/2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3)/2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S	degree of saturation
SL	shrinkage limit
s_u	undrained shear strength
T	time factor for consolidation
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

APPENDIX C

Calculations

Bearing Resistance on Bedrock for Abutments

PROBLEM STATEMENT & OBJECTIVE

Calculate the strength, service, and extreme limit state bearing resistance for the abutments 1 and 2.

EXECUTIVE SUMMARY

A factored bearing resistance of	25	ksf for the strength limit state is recommended.
A factored bearing resistance of	40	ksf for the service limit state for 1 in. settlement is recommended.
A factored bearing resistance of	44	ksf for the extreme event limit state is recommended.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
2. NCHRP Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, 2010.
3. Hunt, Roy E., Geotechnical Engineering Analysis and Evaluation, 1986.

AVAILABLE INFORMATION

1. Published information for bedrock in Brandon, Vermont area from the United States Geological Survey website.
2. Preliminary bearing pressure data provided by CLD:
    - -Abutment 1 strength bearing pressure 1132 kPa (23.6 ksf)
    - -Abutment 1 service bearing pressure 802 kPa (16.75 ksf)
    - -Abutment 2 strength bearing pressure 717 kPa (15 ksf)
    - -Abutment 2 service bearing pressure 507 kPa (10.58 ksf)
3. Photo of exposed rock outcrop at bridge taken by Haley & Aldrich.
4. Haley & Aldrich boring logs from August 2015.

ASSUMPTIONS

1. The footings for the abutments will bear on bedrock.
2. Bedrock type in area is quartzite or dolomite.
3. Bedrock compressive strength is 490 to 1400 tsf (980 to 2800 ksf) based on reference.
4. Conditions of bedrock joints are based on observations in photograph.

PROCEDURE FOR STRENGTH LIMIT STATE

Excerpt from AASHTO LRFD 2014:

10.6.3.2 - Bearing Resistance of Rock

10.6.3.2.1 - General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profile, and other similar profiles as they apply at a particular site. For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock, and the orientation and condition of the discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

10.6.3.2.2 Semiemprical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geometrics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures. The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

C. 10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on the rock mass quality measured in terms of RMR system, the nominal bearing resistance of a rock mass varies from small fraction to six times the unconfined compressive strength of intact rock core samples.

Nominal bearing resistance equation based on Carter and Kulhawy (1988) from NCHRP Report 651:

$q_n = q_u(\sqrt{s}+(m\sqrt{s} + s)^{0.5})$ Equation 82b An errata to Carter and Kulhawy 1988

Rock Mass Ratio (RMR) and strength parameters m and s to be used in Equation 82b from NCHRP Report 651:

RMR using Tables 15 and 16

m and s using Tables 17 and 19

Resistance factor ϕ from Table 10.5.5.2.2-1 in AASHTO LRFD 2014 for bearing resistance of footings on rock

PROCEDURE FOR SERVICE LIMIT STATE

Excerpt from AASHTO LRFD 2014:

10.6.2.6 - Bearing Resistance at the Service Limit State

10.6.2.6.1 - Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

See Table C10.6.2.6.1-1 Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Use AASHTO LRFD 2014 presumptive bearing resistance for service limit state for settlement stated.

PROCEDURE FOR EXTREME EVENT LIMIT STATE

Excerpt from AASHTO LRFD 2014:

11.5.8 - Resistance Factors for Extreme Event Limit state

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state. For overall stability of the retaining wall when earthquake loading is included, a resistance factor ϕ shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSEWalls.

Use nominal resistance calculated for the Strength Limit State and apply a resistance factor of 0.8 from AASHTO LRFD 2014 Section 11.5.8 to obtain the factored resistance.

CALCULATIONS - STRENGTH LIMIT STATE

Table 15 from NCHRP Report 651:

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, unconfined compressive test is preferred		
		Unconfined compressive strength	>4,320 ksf	2,160–4,320 ksf	1,080–2,160 ksf	520–1,080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft	3–10 ft	1–3 ft	2 in–1 ft	<2 in		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in thick or • Joints open 0.05–0.2 in • Continuous joints	• Soft gouge >0.2 in thick or • Joints open >0.2 in • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal/hr	400–2,000 gal/hr	>2,000 gal/hr			
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

Table 16 from NCHRP Report 651:

Strike and dip orientations of joints		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Total RMR Rating Calculation:

Parameter	Value	Relative Rating	
Intact Rock Strength	1,080–2,160 ksf	7	Geology and Typical Rock Strength
RQD	50% to 75%	13	Estimated from photos
Joint Spacing	2 in–1 ft	10	Estimated from photos
Joint Condition	Slightly rough surfaces Separation <0.05 in Soft joint wall rock	12	Estimated from photos
Groundwater Conditions	Water under moderate pressure	4	
Joint Strike and Dip	Unfavorable	–15	
	Total Rating =	46	

CALCULATIONS - STRENGTH LIMIT STATE

Table 19 from NCHRP Report 651:

Rock quality	Constants	Rock type A = Carbonate rocks with well developed crystal cleavage— *dolomite, limestone, and marble* B = Lithified argillaceous rocks—*mudstone, siltstone, shale, and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polymineralic igneous crystalline rocks— *andesite, dolerite, diabase, and rhyolite* E = Coarse-grained polymineralic igneous and metamorphic crystalline rocks—*amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100	m s	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft. CSIR rating: *RMR* = 85	m s	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft. CSIR rating: *RMR* = 65	m s	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft. CSIR rating: *RMR* = 44	m s	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	m s	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced < 2 in with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	m s	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

Table 17 from NCHRP Report 651:

RMR rating	100–81	80–61	60–41	40–21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

s and m Calculation:

Rock Quality	Rock Type	Rock Type Description	m	s
Fair	A	Carbonate rocks with well developed crystal cleavage - dolomite, limestone, and marble.	0.128	0.00009

Semi-empirical method by Carter and Kulhawy 1988:

q_u =	8,333	psi	from Hunt reference
q_n =	55	ksf	Equation 82b
ϕ =	0.45		from Table 10.5.5.2.2-1
q_R =	25	ksf	Equation 82b

CALCULATIONS - SERVICE LIMIT STATE

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

Based on Table C10.6.2.6.1-1 the service limit state for bearing resistance on schist bedrock is recommended at
40 ksf for settlements of 1 in.

CALCULATIONS - EXTREME LIMIT STATE

From the Strength Limit State calculations, the nominal bearing resistance is the following:

q_{ult} = 55 ksf

Using a resistance factor of 0.8 from Section 11.5.8, the factored bearing resistance is the following:

q_R = 44 ksf

CALCULATIONS		File No.	41107-200
		Sheet	7 of 7
Client	CLD Consulting Engineers	Date	16-Jun-17
Project	Bridge No. 114, Us Route 7 over Neshobe River, Brandon, Vermont	Computed by	MMH
Subject	Bearing Resistance for Abutments 1 and 2	Checked by	JGD

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The recommended factored bearing resistance for the strength limit state is 25 ksf

Service Limit State

The recommended presumptive value for schist bedrock is 40 ksf for the service limit state for a settlement up to 1 in.

Extreme Event Limit State

The recommended factored bearing resistance for the extreme event limit state is 44 ksf

LIMITATIONS

AASHTO

LRFD BRIDGE DESIGN SPECIFICATIONS

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

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THE VOICE OF TRANSPORTATION

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Part II: Sections 7—Index

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Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Articl… and a comparison of those resistance factors t… Allowable Stress Design practice, e.g., … (2002), is provided in Allen (2005).

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

Scour design for the design flood must … requirement that the factored foundation resi… scour is greater than the factored load deter… the scoured soil removed. The resistance fac… those used in the Strength Limit State, witho…

10.5.5.2.2—Spread Footings

C10.5.5.2.2

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength L…

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

$$S_s = C_{\alpha\varepsilon} H_c \log\left(\frac{t_2}{t_1}\right) \qquad (10.6.2.4.3\text{-}10)$$

where:

H_c = initial height of compressible soil layer (ft)

e_o = void ratio at initial vertical effective stress (dim)

t_1 = time when secondary settlement begins, i.e., typically at a time equivalent to 90 percent average degree of primary consolidation (yr)

t_2 = arbitrary time that could represent the service life of the structure (yr)

C_α = secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (dim)

$C_{\alpha\varepsilon}$ = modified secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (dim)

indicates preloading and surcharging may not be effective in eliminating secondary compression.

10.6.2.4.4—Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system and designed in accordance with the provisions of this Section, elastic settlements may generally be assumed to be less than 0.5 in. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics shall be made.

Where rock is broken or jointed (relative rating of ten or less for *RQD* and joint spacing), the rock joint condition is poor (relative rating of ten or less) or the criteria for fair to very good rock are not met, a settlement analysis should be conducted, and the influence of rock type, condition of discontinuities, and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock, in feet, should be taken as:

- For circular (or square) footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{rI_p}{144\,E_m} \qquad (10.6.2.4.4\text{-}1)$$

in which:

C10.6.2.4.4

In most cases, it is sufficient to determine settlement using the average bearing stress under the footing.

Where the foundations are subjected to a very large load or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus, E_m. In some cases, the value of E_m can be estimated through empirical correlation with the value of the modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in-situ tests, such as plate loading and pressuremeter tests.

$$I_p = \frac{\left(\sqrt{\pi}\right)}{\beta_z} \qquad (10.6.2.4.4\text{-}2)$$

- For rectangular footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{144\,E_m} \qquad (10.6.2.4.4\text{-}3)$$

in which:

$$I_p = \frac{\left(L/B\right)^{1/2}}{\beta_z} \qquad (10.6.2.4.4\text{-}4)$$

where:

q_o = applied vertical stress at base of loaded area (ksf)

ν = Poisson's Ratio (dim)

r = radius of circular footing or $B/2$ for square footing (ft)

I_p = influence coefficient to account for rigidity and dimensions of footing (dim)

E_m = rock mass modulus (ksi)

β_z = factor to account for footing shape and rigidity (dim)

Values of I_p should be computed using the β_z values presented in Table 10.6.2.4.2-1 for rigid footings. Where the results of laboratory testing are not available, values of Poisson's ratio, ν, for typical rock types may be taken as specified in Table C10.4.6.5-2. Determination of the rock mass modulus, E_m, should be based on the methods described in Sabatini (2002).

The magnitude of consolidation and secondary settlements in rock masses containing soft seams or other material with time-dependent settlement characteristics should be estimated by applying procedures specified in Article 10.6.2.4.3.

10.6.2.5—Overall Stability

Overall stability of spread footings shall be investigated using Service I Load Combination and the provisions of Articles 3.4.1, 10.5.2.3, and 11.6.3.4.

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

C10.6.2.6.1

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

10.6.3—Strength Limit State Design

10.6.3.1—Bearing Resistance of Soil

10.6.3.1.1—General

Bearing resistance of spread footings shall be determined based on the highest anticipated position of groundwater level at the footing location.

The factored resistance, q_R, at the strength limit state shall be taken as:

$$q_R = \varphi_b \, q_n \qquad (10.6.3.1.1\text{-}1)$$

where:

φ_b = resistance factor specified in Article 10.5.5.2.2

q_n = nominal bearing resistance (ksf)

Where loads are eccentric, the effective footing dimensions, L' and B', as specified in Article 10.6.1.3, shall be used instead of the overall dimensions L and B in all equations, tables, and figures pertaining to bearing resistance.

C10.6.3.1.1

The bearing resistance of footings on soil should be evaluated using soil shear strength parameters that are representative of the soil shear strength under the loading conditions being analyzed. The bearing resistance of footings supported on granular soils should be evaluated for both permanent dead loading conditions and short-duration live loading conditions using effective stress methods of analysis and drained soil shear strength parameters. The bearing resistance of footings supported on cohesive soils should be evaluated for short-duration live loading conditions using total stress methods of analysis and undrained soil shear strength parameters. In addition, the bearing resistance of footings supported on cohesive soils, which could soften and lose strength with time, should be evaluated for permanent dead loading conditions using effective stress methods of analysis and drained soil shear strength parameters.

The position of the groundwater table can significantly influence the bearing resistance of soils through its effect on shear strength and unit weight of the foundation soils. In general, the submergence of soils will reduce the effective shear strength of cohesionless (or granular) materials, as well as the long-term (or drained) shear strength of cohesive (clayey) soils. Moreover, the effective unit weights of submerged soils are about half of those for the same soils under dry conditions. Thus, submergence may lead to a significant reduction in the bearing resistance provided by the foundation soils, and it is essential that the bearing resistance analyses be carried out under the assumption of the highest groundwater table expected within the service life of the structure.

Footings with inclined bases should be avoided wherever possible. Where use of an inclined footing base cannot be avoided, the nominal bearing resistance determined in accordance with the provisions herein should be further reduced using accepted corrections for inclined footing bases in Munfakh, et al. (2001).

Because the effective dimensions will vary slightly for each limit state under consideration, strict adherence to this provision will require re-computation of the nominal bearing resistance at each limit state.

Further, some of the equations for the bearing resistance modification factors based on L and B were

$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

D_f = footing embedment depth taken to the bottom of the footing (ft)

The nominal bearing resistance, in ksf, for footings on cohesionless soils based on *CPT* results may be taken as:

$$q_n = \frac{\overline{q_c} B}{40}\left(C_{wq}\frac{D_f}{B} + C_{w\gamma}\right) \qquad (10.6.3.1.3\text{-}2)$$

where:

$\overline{q_c}$ = average cone tip resistance within a depth range B below the bottom of the footing (ksf)

B = footing width (ft)

$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

D_f = footing embedment depth taken to the bottom of the footing (ft)

10.6.3.1.4—Plate Load Tests

The nominal bearing resistance may be determined by plate load tests, provided that adequate subsurface explorations have been made to determine the soil profile below the foundation. Where plate load tests are conducted, they should be conducted in accordance with AASHTO T 235 and ASTM D1194.

The nominal bearing resistance determined from a plate load test may be extrapolated to adjacent footings where the subsurface profile is confirmed by subsurface exploration to be similar.

C10.6.3.1.4

Plate load tests have a limited depth of influence and furthermore may not disclose the potential for long-term consolidation of foundation soils.

Scale effects should be addressed when extrapolating the results to performance of full scale footings. Extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided herein for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification, i.e., layer thicknesses, depths, and properties. Plate load test results should be applied only within a sub-area of the project site for which the subsurface conditions, i.e., stratification, geologic history, and properties, are relatively uniform.

10.6.3.2—Bearing Resistance of Rock

10.6.3.2.1—General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and *RQD* may be applicable. For footings on less competent rock, more detailed investigations and

C10.6.3.2.1

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

The design procedures for foundations in rock have been developed using the RMR, rock mass rating system. Classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

10.6.3.2.2—Semiempirical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geomechanics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures.

The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

C10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on rock mass quality measured in terms of *RMR* system, the nominal bearing resistance of a rock mass varies from a small fraction to six times the unconfined compressive strength of intact rock core samples.

10.6.3.2.3—Analytic Method

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be considered.

C10.6.3.2.3

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing resistance for each of the failure modes can be found in Kulhawy and Goodman (1987), Goodman (1989), and Sowers (1979).

10.6.3.2.4—Load Test

Where appropriate, load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-third of the corresponding footing dimension, *B* or *L*, for footings on soils, or 0.45 of the corresponding footing dimensions *B* or *L*, for footings on rock.

C10.6.3.3

A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of $B/3$ were comparable to those of ASD with an eccentricity of $B/6$. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of $B/2$ would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.

Table 11.5.7-1—Resistance Factors for Permanent Retaining Walls

Wall-Type and Condition		Resistance Factor
Nongravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		0.75
Pullout resistance of anchors [1]	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 [1] 0.70 [1] 0.50 [1]
Pullout resistance of anchors [2]	• Where proof tests are conducted	1.0 [2]
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A615 bars) • High strength steel (e.g., ASTM A722 bars)	0.90 [3] 0.80 [3]
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semigravity Walls		
Bearing resistance	• Gravity and semigravity walls • MSE walls	0.55 0.65
Sliding		1.0
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements [4] • Static loading Grid reinforcements [4] [5] • Static loading	 0.75 0.65
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading	0.90
Pullout resistance of tensile reinforcement	• Static loading	0.90
Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

(1) Apply to presumptive ultimate unit bond stresses for preliminary design only in Article C11.9.4.2.

(2) Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored load on the anchor.

(3) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel apply the resistance factor to guaranteed ultimate tensile strength.

(4) Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

(5) Applies to grid reinforcements connected to a rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

11.5.8—Resistance Factors—Extreme Event Limit State

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state.

For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ, of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE walls.

For tensile resistance of metallic reinforcement and connectors, when earthquake loading is included, the following resistance factors shall be used:

- Strip reinforcements, $\phi = 1.0$
- Grid reinforcement, $\phi = 0.85$

Table 11.5.7-1 Notes 4 and 5 also apply to these resistance factors for metallic reinforcements.

For tensile resistance of geosynthetic reinforcement and connectors, a resistance factor, ϕ, of 1.20 shall be used.

For pullout resistance of metallic and geosynthetic reinforcement, a resistance factor, ϕ, of 1.20 shall be used.

C11.5.8

A resistance factor of 1.0 is recommended for the extreme event limit state in view of the unlikely occurrence of the loading associated with the design earthquake. The choice of 1.0 is influenced by the following factors:

- For competent soils that are not expected to lose strength during seismic loading (e.g., due to liquefaction of saturated cohesionless soils or strength reduction of sensitive clays), the use of static strengths for seismic loading is usually conservative, as rate-of-loading effects tend to increase soil strength for transient loading.
- Earthquake loads are transient in nature and hence, if soil yield occurs, the net effect is an accumulated small deformation as opposed to foundation failure. This assumes that global stability is adequate.

Using a resistance factor of 1.0 for soil assumes ductile behavior. While this is a correct assumption for many soils, it is inappropriate for brittle soils where there is a significant post-peak strength loss (e.g., stiff over-consolidated clays, sensitive soils). For such conditions, special studies will be required to determine the appropriate combination of resistance factor and soil strength.

For bearing resistance, a slightly lower resistance factor of 0.8 is recommended for gravity and semigravity walls and 0.9 for MSE walls to reduce the possibility that a bearing resistance failure could occur before the wall moves laterally in sliding, reducing the likelihood of excessive wall tilting or collapse, consistent with the design objective of no collapse.

11.6—ABUTMENTS AND CONVENTIONAL RETAINING WALLS

11.6.1—General Considerations

11.6.1.1—General

Rigid gravity and semigravity retaining walls may be used for bridge substructures or grade separation and are generally for permanent applications.

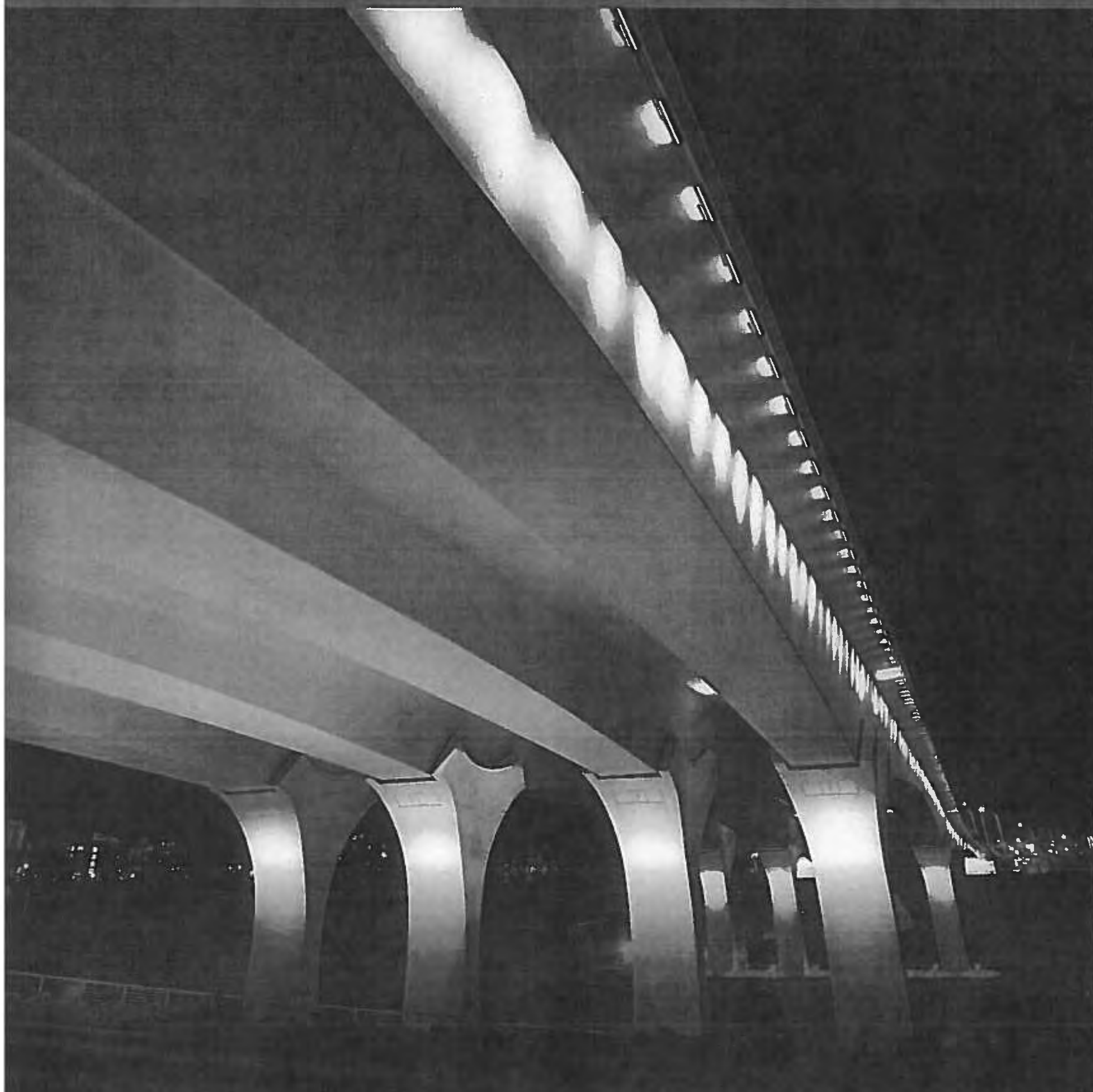
Rigid gravity and semigravity walls shall not be used without deep foundation support where the bearing soil/rock is prone to excessive total or differential settlement.

C11.6.1.1

Conventional retaining walls are generally classified as rigid gravity or semigravity walls, examples of which are shown in Figure C11.6.1.1-1. These types of walls can be effective for both cut and fill wall applications.

Excessive differential settlement, as defined in Article C11.6.2.2 can cause cracking, excessive bending or shear stresses in the wall, or rotation of the wall structure.

AASHTO LRFD BRIDGE



DESIGN SPECIFICATIONS

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Table 10.4.6.4-1—Geomechanics Classification of Rock Masses

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft	3–10 ft	1–3 ft	2 in.–1 ft	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)

Rock Quality	Constants	Rock Type A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polyminerallic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite* E = Coarse grained polymineralic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100	*m* *s*	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft CSIR rating: *RMR* = 85	*m* *s*	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft CSIR rating: *RMR* = 65	*m* *s*	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft CSIR rating: *RMR* = 44	*m* *s*	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	*m* *s*	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	*m* *s*	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

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NATIONAL
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REPORT 651

LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

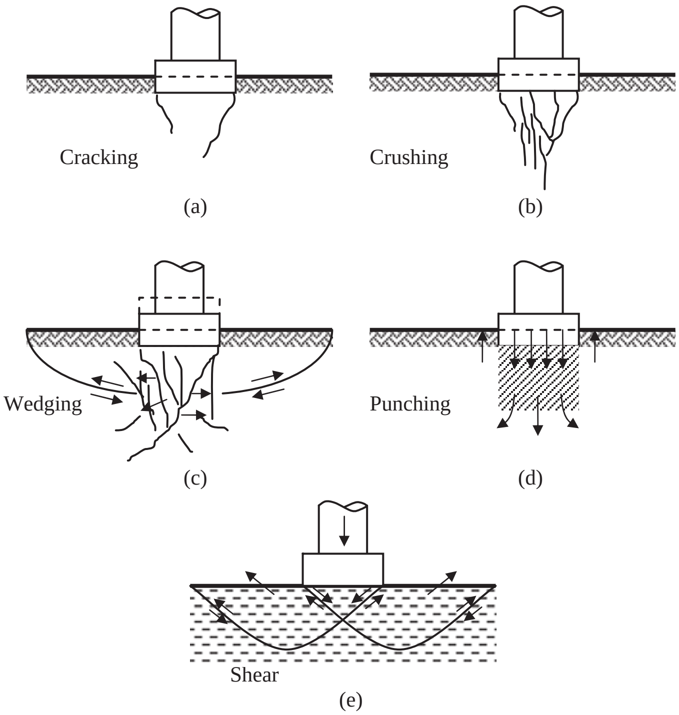

Figure 36. Modes of failure of a footing on rock including development of failure through crack propagation and crushing beneath the footing (a-c), punching through collapse of voids (d), and shear failure (e) (based on Goodman, 1989).

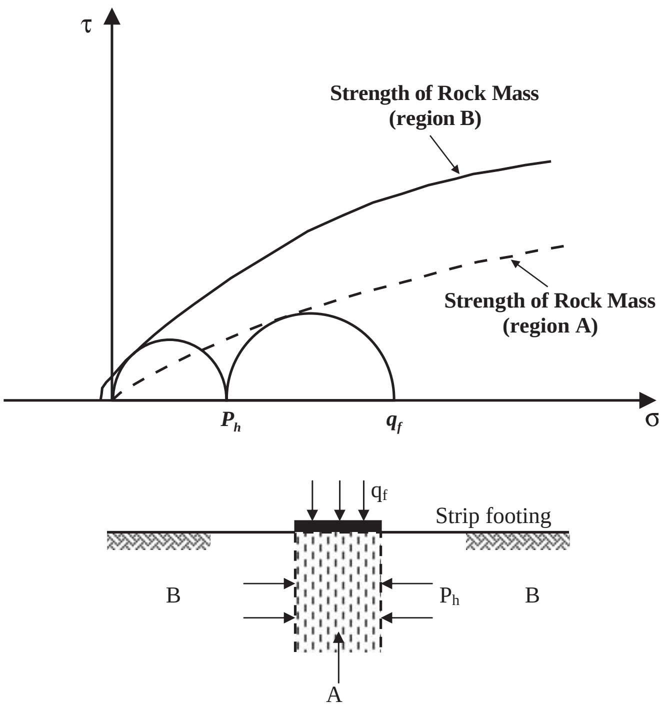

Figure 37. Analysis of bearing capacity on rock (based on Goodman, 1989).

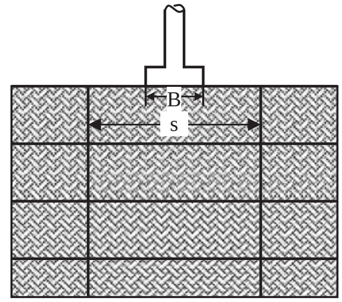

Figure 38. Footing on rock with open, vertical joints (based on Goodman, 1989).

Comparing the results of Goodman's (1989) computations with Equations 79 and 80 shows that open joints reduce the bearing capacity only when the ratio S/B is in the range from 1 to 5. The bearing capacity of footings on rock with open joints increases with increasing ϕ_f for any of the S/B ratios ranging from 1 to 5.

1.7.6 Carter and Kulhawy (1988)

Carter and Kulhawy (1988) suggested that the Hoek and Brown strength criterion for jointed rock masses (Hoek and Brown, 1980, see also Section 1.8.2.4) can be used in the evaluation of bearing capacity. The curved strength envelope for jointed rock mass can be expressed as

$$\sigma_1 = \sigma_3 + \left(mq_u\sigma_3 + sq_u^2\right)^{0.5} \tag{81}$$

where

σ_1 = major principal effective stress,
σ_3 = minor principal effective stress,
q_u = uniaxial compressive strength of the intact rock.
s and m = empirically determined strength parameters for the rock mass, which are to some degree analogous to c and ϕ_f of the Mohr-Coulomb failure criterion.

Carter and Kulhawy (1988) suggested that an analysis of the bearing capacity of a rock mass obeying this criterion can be made using the same approximate technique as used in the Bell (1915) solution. The details of this approach are described in Figure 39. A lower bound to the failure load was calculated by finding a stress field that satisfies both equilibrium and the failure criterion. For a strip footing, the rock mass beneath the foundation may be divided into two zones with homogeneous stress conditions at failure throughout each, as shown in Figure 39. The vertical stress in Zone I is assumed to be zero, while the horizontal stress is equal to the uniaxial compressive strength of the rock mass, given by Equation 81 as $s^{0.5}q_u$. For equilibrium, continuity of the horizontal stress

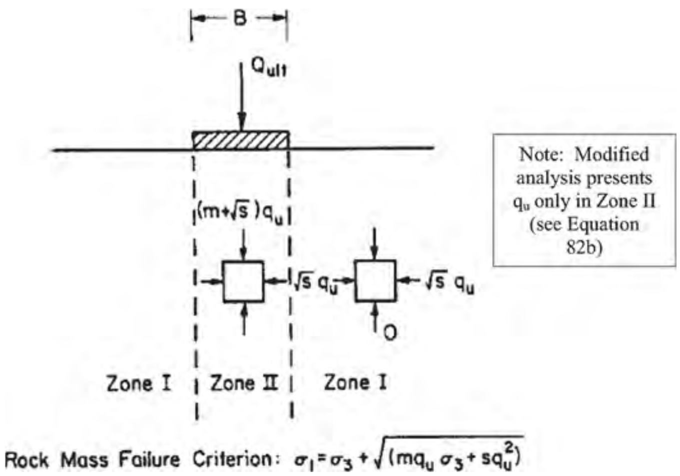

Figure 39. Lower bound solution for bearing capacity (Carter and Kulhawy, 1988).

across the interface must be maintained and therefore the bearing capacity of the strip footing may be evaluated from Equation 81 (with $\sigma_3 = s^{0.5}q_u$) as

$$q_{ult} = \left(m+\sqrt{s}\right)q_u \tag{82a}$$

In an errata to Carter and Kulhawy (1988), Equation (82a) was modified to the following:

$$q_{ult} = \left(\sqrt{s}+\left(m\sqrt{s}+s\right)^{0.5}\right)q_u \tag{82b}$$

A similar approach to the bearing capacity analysis of a strip footing was proposed by Carter and Kulhawy (1988) to be used for a circular foundation with an interface between the two zones that was a cylindrical surface of the same diameter as the foundation. In this axisymmetric case, the radial stress transmitted across the cylindrical surface at the point of collapse of the foundation may be greater than $q_u\sqrt{s}$, without necessarily violating either radial equilibrium or the failure criterion. However, because of the uncertainty of this value, the radial stress at the interface is also assumed to be $q_u\sqrt{s}$ for the case of a circular foundation. Therefore, the predicted (lower bound) bearing capacity is given by Equations 82a and 82b. The m and s constants are determined by the rock type and the conditions of the rock mass, and selecting an appropriate category is easier if either the Rock Mass Rating (RMR) system or the Geological Strength Index (GSI) classification data are available as outlined below. Both bearing capacity formulations expressed in Equations 82a and 82b were investigated in this study.

1.8 Rock Classification and Properties

1.8.1 Overview

A rock mass comprises blocks of intact rock that are separated by discontinuities such as cleavage, bedding planes, joints, and faults. Table 8 provides a summary of rock mass discontinuity definitions and characteristics. These naturally formed discontinuities create weakness surfaces within the rock mass, thereby reducing the material strength. As previously discussed, the influence of the discontinuities upon the material strength depends upon the scale of the foundation relative to the position and frequency of the discontinuities (Canadian Foundation Geotechnical Society, 2006).

This section provides a short review of rock mass classification/characterization systems and rock properties that are relevant to the methods selected for bearing capacity evaluation. Methods allowing engineering classification of rock mass are reviewed including the Rock Mass index (RM*i*) system, RMR system and the Hoek-Brown GSI.

1.8.2 Engineering Rock Mass Classification

1.8.2.1 Classification Methods

A number of classification systems have been developed to provide the basis for engineering characterization of rock masses. A comprehensive overview of this subject is provided by Hoek et al. (1995). Most of the classification systems incorporating various parameters were derived from civil engineering case histories in which all components of the engineering geological parameters of the rock mass were considered (Wickham et al., 1972; Bieniawski, 1973, 1979, 1989; Barton et al., 1974). More recently, the systems have been modified to account for the conditions affecting rock mass stability in underground mining. While no single classification system has been developed for or applied to foundation design, the type of information collected for the two more common civil engineering classification schemes—the Q system (Barton et al., 1974), used in tunnel design, and RMR (Bieniawski, 1989), used in tunnel and foundation design—are often considered. These techniques have been applied to empirical design situations, where previous experience greatly affects the design of the excavation in the rock mass. Table 9 outlines the many classification systems and their uses. Detailed descriptions of the different systems and the engineering properties associated with them are beyond the scope of this work and are restricted to the methods relevant to the current research.

The two most commonly used rock mass classification systems today are RMR, developed by Bieniawski (1973) and

McGRAW-HILL SERIES IN GEOTECHNICAL ENGINEERING

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Hunt **Geotechnical Engineering Investigation Manual** (1984)
Hunt **Geotechnical Engineering Analysis and Evaluation** (1986)

GEOTECHNICAL ENGINEERING ANALYSIS AND EVALUATION

ROY E. HUNT

Consulting Engineer

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TABLE 3.12
COMMON ROCK TYPES AND TYPICAL ENGINEERING PROPERTIES*

Rock type†	Texture	Fabric structure	γ_d, g/cm²	U_c, tsf, kg/cm²	E_r 10⁴ tsf, kg/cm²
		IGNEOUS			
Granite	Coarse to medium	Massive relatively tight, and widely spaced joints	2.69	700–1750	28–49
Diorite	Coarse to medium		2.82	700–1750	35–56
Gabbro	Coarse to medium		2.88	1050–2100	49–84
Rhyolite	Fine	Massive, extensive jointing, often vesicular	2.59	700–1750	35–56
Andesite	Fine		2.66	700–1750	42–63
Basalt	Fine		2.85	1050–2100	49–90
Obsidian	Glassy	Massive, continuous	2.20	140–560	7–28
Tuff	Coarse	Cemented ash, porous	1.60	14–70	1–7
		METAMORPHIC			
Gneiss	Coarse to medium	Banded to foliated	2.70	700 1400	28–56
Schist	Fine	Foliated	2.67	350–1050	14–35
Slate	Fine	Platy	2.69	700–1400	35–56
Quartzite	Fine	Massive, fine and widely spaced joints	2.66	1050–2450	42–56
Marble	Fine to very fine		2.69	840–2100	49–70
Serpentine	Various	Massive, often soft	2.53	70– 700	7–35
		SEDIMENTARY			
Conglomerate	Coarse, rounded	Layered, cemented	2.48	350–1050	7–35
Breccia	Course, angular	Layered, cemented	2.53	350–1050	7–35
Sandstone	Medium	Layered, cemented	2.35	280– 840	7–21
Siltstone	Fine	Layered, cemented	1.8–2.4	7– 350	3–14
Shale‡	Very fine	Laminated, compaction shales unstable, cemented shales stable	1.6–2.2	7– 350	3–14
Limestone	Fine	Massive, stratified, soluble, cavities form	2.64	350–1050	14–42
Dolomite	Fine	Massive, some recrystallization	2.67	490–1400	28–56

*After NAVFAC (1971).[44] Properties are for sound, unweathered specimens without voids or fractures, tested dry in the laboratory. Elasticity and strength depend on porosity, cementation, and in foliated, platy, or laminated rocks, on loading direction. Saturated values for U_c and E_r are usually 80 to 90% of the dry values given.

†For detailed descriptions of rock types, composition, textures, fabrics, and structure, see Hunt (1984).[1]

‡See also Table 3.30 of Hunt (1984).[1]

Based on published bedrock data in the area of Brandon, Vermont - quartzite and dolomite formations. Assume weaker formation (dolomite) for calculations.

Bearing Resistance on Soil
for East and West Retaining Walls

PROBLEM STATEMENT & OBJECTIVE

Calculate the strength and service limit state bearing resistance for the new east retaining wall.

EXECUTIVE SUMMARY

1. The factored bearing resistance at the strength limit state is about	3.5	ksf.
2. The factored bearing resistance at the service limit state for 1 in. of settlement is about	3.0	ksf.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

AVAILABLE INFORMATION

1. Haley & Aldrich boring logs from August 2015.
2. Loading provided by CLD on 19 June 2017.
3. Plan sheet 7 from the preliminary plan set titled "Proposed Improvement, Bridge Project, Town of Brandon, County of Rutland, Us route 7 (Principal Arterial), Bridge No. 114" dated 26 June 2016.
4. Email from CLD dated 21 June 2017 with footing length.
5. Strength bearing load provided by CLD is 459 psf and service bearing load provided by CLD is 313 psf.

ASSUMPTIONS

1. Bearing material unknown, new footing will bear on same footprint as old footing so assume medium dense granular material.

PROCEDURE FOR STRENGTH LIMIT STATE

10.6.3.1.2a- Basic Formulation for Nominal Bearing Resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$$ Equation 10.6.3.1.2a-1

$$N_{cm} = N_c s_c i_c$$ Equation 10.6.3.1.2a-2

$$N_{qm} = N_q s_q d_q i_q$$ Equation 10.6.3.1.2a-3

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ Equation 10.6.3.1.2a-4

q_n =	nominal strength limit state bearing resistance (ksf)
RF =	resistance factor from Table 10.5.5.2.2-1
q_R =	factored strength limit state bearing resistance (ksf)
c =	cohesion, taken as undrained shear strength (ksf)
N_c =	cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
N_q =	surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
N_γ =	unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
γ =	total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

HALEY ALDRICH	CALCULATIONS	File No.	41107-200
		Sheet	2 of 4
Client	CLD Consulting Engineers, Inc.	Date	20-Jun-17
Project	Bridge No. 114, US Route 7 over Neshobe River, Brandon, Vermont	Computed by	MMH
Subject	Bearing Resistance for East Retaining Wall	Checked by	JGD

D_f =	footing embedment depth (ft)
D_w =	depth of water below bottom of footing (ft)
B =	footing width (ft)
e_B =	footing width eccentricity (ft) as specified in Section 10.6.3.3
B' =	effective footing width (B-2e) (ft)
L =	footing length (ft)
e_L =	footing length eccentricity (ft) as specified in Section 10.6.3.3
L' =	effective footing length (L-2e) (ft)
C_{wq},$C_{w\gamma}$ =	correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
s_c,s_q,s_γ =	footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)
d_q =	correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4 (dim).
i_c,i_q,i_γ =	load inclination factors

CALCULATION FOR STRENGTH LIMIT STATE

B =	8.2	ft	from plan set provided by CLD
e_B =	0.6	ft	from loading information provided by CLD
B' =	7.0	ft	
L =	42.65	ft	from CLD email
e_L =	0	ft	
L'=	42.65	ft	
c =	0	ksf	
γ =	115	pcf	
ϕ =	28	degrees	
D_w	0	ft	
D_f	5	ft	from plan set provided by CLD
$N_f = f(\phi)$	2.77		
C_{wq}	0.5		
$C_{w\gamma}$	0.5		
N_c	25.8		
S_c	1.09		
i_c	1		
N_{cm}	28.2		
N_q	14.7		
S_q	1.09		
d_q	1		
i_q	1		
N_{qm}	16.0		
N_γ	16.7		
S_γ	0.93		
i_γ	1		
$N_{\gamma m}$	15.6		
q_n	7.7	ksf	
RF	0.45		
q_R	3.5	ksf	

HALEY ALDRICH	CALCULATIONS	File No.	41107-200
		Sheet	3 of 4
Client	CLD Consulting Engineers, Inc.	Date	20-Jun-17
Project	Bridge No. 114, US Route 7 over Neshobe River, Brandon, Vermont	Computed by	MMH
Subject	Bearing Resistance for East Retaining Wall	Checked by	JGD

PROCEDURE FOR SERVICE LIMIT STATE

10.6.2.4.2 - Settlement of Footings on Cohesionless Soils

$$S_e = \frac{q_0(1-v^2)\sqrt{A'}}{144\,E_s\beta_z}$$ Equation 10.6.2.4.2-1

where

q_0 = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of Es are not available from the results of insitu or laboratory tests (ksi)

β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

v = Poisson's Ratio, taken as specified in Article 10.4.6.2 if direct measurements of v are not available from the results of insitu or laboratory tests (dim)

S_e = elastic settlement (ft)

CALCULATION FOR SERVICE LIMIT STATE

A =	350	ft^2
E_s =	4	ksi, Table C10.4.6.3-1
v =	0.3	
β_z =	1.22	Table 10.6.2.4.2-1
S_e =	0.083	ft

q_0 = 3 ksf

Compare to presumptive bearing resistances in AASHTO LRFD:

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

C10.6.2.6.1

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

Recommended presumptive values

3 ksf for SP and SM soils

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The factored bearing resistance for the strength limit state is 3.5 ksf

Service Limit State

The factored bearing resistance for the service limit state is 3.0 ksf for a 1 in. settlement.

The bearing pressure for the strength and service limit state provided by CLD is less than the recommended bearing resistances.

Client	CLD Consulting Engineers, Inc.	Date	21-Jul-17
Project	Bridge No. 114, US Route 7 over Neshobe River, Brandon, Vermont	Computed by	MMH
Subject	Bearing Resistance for West Retaining Wall	Checked by	JGD

PROBLEM STATEMENT & OBJECTIVE

Calculate the strength and service limit state bearing resistance for the new west retaining wall.

EXECUTIVE SUMMARY

1. The factored bearing resistance at the strength limit state is about 2.8 ksf.
2. The factored bearing resistance at the service limit state for 1 in. of settlement is about 3.0 ksf.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

AVAILABLE INFORMATION

1. Haley & Aldrich boring logs from August 2015.
2. Loading provided by CLD on 20 June 2017.
3. Plan sheet 7 from the preliminary plan set titled "Proposed Improvement, Bridge Project, Town of Brandon, County of Rutland, Us route 7 (Principal Arterial), Bridge No. 114" dated 26 June 2016.
4. Email from CLD dated 21 June 2017 with footing length.
4. Strength bearing load provided by CLD is 270 psf and service bearing load provided by CLD is 209 psf.

ASSUMPTIONS

1. Bearing material unknown, new footing will bear on same footprint as old footing so assume medium dense granular material.

PROCEDURE FOR STRENGTH LIMIT STATE

10.6.3.1.2a- Basic Formulation for Nominal Bearing Resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$$ Equation 10.6.3.1.2a-1

$$N_{cm} = N_c s_c i_c$$ Equation 10.6.3.1.2a-2

$$N_{qm} = N_q s_q d_q i_q$$ Equation 10.6.3.1.2a-3

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$$ Equation 10.6.3.1.2a-4

q_n = nominal strength limit state bearing resistance (ksf)
RF = resistance factor from Table 10.5.5.2.2-1
q_R = factored strength limit state bearing resistance (ksf)
c = cohesion, taken as undrained shear strength (ksf)
N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
γ = total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

D_f = footing embedment depth (ft)
D_w = depth of water below bottom of footing (ft)
B = footing width (ft)
e_B = footing width eccentricity (ft) as specified in Section 10.6.3.3
B' = effective footing width (B-2e) (ft)
L = footing length (ft)
e_L = footing length eccentricity (ft) as specified in Section 10.6.3.3
L' = effective footing length (L-2e) (ft)
C_{wq},$C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
s_c,s_q,s_γ = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)
d_q = correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4 (dim).
i_c,i_q,i_γ = load inclination factors

CALCULATION FOR STRENGTH LIMIT STATE

B =	8.2	ft	from plan set provided by CLD
e_B =	0.1	ft	from loading information provided by CLD
B' =	8.0	ft	
L =	9.2	ft	from CLD email
e_L =	0	ft	
L'=	9.2	ft	
c =	0	ksf	
γ =	115	pcf	
ϕ =	28	degrees	
D_w	0	ft	
D_f	3	ft	
$N_f = f(\phi)$	2.77		
C_{wq}	0.5		
$C_{w\gamma}$	0.5		
N_c	25.8		
S_c	1.50		
i_c	1		
N_{cm}	38.6		
N_q	14.7		
S_q	1.46		
d_q	1		
i_q	1		
N_{qm}	21.5		
N_γ	16.7		
S_γ	0.65		
i_γ	1		
$N_{\gamma m}$	10.9		
q_n	6.2	ksf	
RF	0.45		
q_R	2.8	ksf	

PROCEDURE FOR SERVICE LIMIT STATE

10.6.2.4.2 - Settlement of Footings on Cohesionless Soils

$$S_e = \frac{q_0 (1 - v^2) \sqrt{A'}}{144\, E_s \beta_z}$$ Equation 10.6.2.4.2-1

where

q_0 = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of Es are not available from the results of insitu or laboratory tests (ksi)

β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

v = Poisson's Ratio, taken as specified in Article 10.4.6.2 if direct measurements of v are not available from the results of insitu or laboratory tests (dim)

S_e = elastic settlement (ft)

CALCULATION FOR SERVICE LIMIT STATE

A =	75	ft^2
E_s =	4	ksi, Table C10.4.6.3-1
v =	0.3	
β_z =	1.07	Table 10.6.2.4.2-1
S_e =	0.083	ft
q_0 =	6	ksf

Compare to presumptive bearing resistances in AASHTO LRFD:

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

C10.6.2.6.1

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

Recommended presumptive values

3 ksf for SP and SM soils

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The factored bearing resistance for the strength limit state is 2.8 ksf

Service Limit State

The factored bearing resistance for the service limit state is 3.0 ksf for a 1 in. settlement.

The bearing pressure for the strength and service limit state provided by CLD is less than the recommended bearing resistances.

AASHTO
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Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim)
Clay: Soft sensitive Medium stiff to stiff Very stiff	 0.347–2.08 2.08–6.94 6.94–13.89	0.4–0.5 (undrained)
Loess Silt	2.08–8.33 0.278–2.78	0.1–0.3 0.3–0.35
Fine Sand: Loose Medium dense Dense	 1.11–1.67 1.67–2.78 2.78–4.17	0.25
Sand: Loose Medium dense Dense	 1.39–4.17 4.17–6.94 6.94–11.11	 0.20–0.36 0.30–0.40
Gravel: Loose Medium dense Dense	 4.17–11.11 11.11–13.89 13.89–27.78	 0.20–0.35 0.30–0.40

Estimating E_s from *SPT N* Value	
Soil Type	E_s (ksi)
Silts, sandy silts, slightly cohesive mixtures	$0.056\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.097\ N1_{60}$
Coarse sands and sands with little gravel	$0.139\ N1_{60}$
Sandy gravel and gravels	$0.167\ N1_{60}$

Estimating E_s from q_c (static cone resistance)	
Sandy soils	$0.028q_c$

The modulus of elasticity for normally consolidated granular soils tends to increase with depth. An alternative method of defining the soil modulus for granular soils is to assume that it increases linearly with depth starting at zero at the ground surface in accordance with the following equation:

$$E_s = nh \times z \tag{C10.4.6.3-1}$$

10.5.5.3.2—Scour

The provisions of Articles 2.6.4.4.2 and 3.7.5 shall apply to the changed foundation conditions resulting from scour. Resistance factors at the strength limit state shall be taken as specified herein. Resistance factors at the extreme event shall be taken as 1.0 except that for uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

C10.5.5.3.2

The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (2004) regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (2005).

10.5.5.3.3—Other Extreme Limit States

Resistance factors for extreme limit state, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

C10.5.5.3.3

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see Article 10.5.5.2.3).

10.6—SPREAD FOOTINGS

10.6.1—General Considerations

10.6.1.1—General

Provisions of this Article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and other substructure and superstructure elements. Special attention shall be given to footings on fill, to make sure that the quality of the fill placed below the footing is well controlled and of adequate quality in terms of shear strength and compressibility to support the footing loads.

Spread footings shall be proportioned and designed such that the supporting soil or rock provides adequate nominal resistance, considering both the potential for adequate bearing strength and the potential for settlement, under all applicable limit states in accordance with the provisions of this Section.

Spread footings shall be proportioned and located to maintain stability under all applicable limit states, considering the potential for, but not necessarily limited to, overturning (eccentricity), sliding, uplift, overall stability and loss of lateral support.

C10.6.1.1

Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor, e.g., soft, wet, frozen, or nondurable, material is used, or if the material is not properly compacted.

Spread footings should not be used on soil or rock conditions that are determined to be too soft or weak to support the design loads without excessive movement or loss of stability. Alternatively, the unsuitable material can be removed and replaced with suitable and properly compacted engineered fill material, or improved in place, at reasonable cost as compared to other foundation support alternatives.

Footings should be proportioned so that the stress under the footing is as nearly uniform as practicable at the service limit state. The distribution of soil stress should be consistent with properties of the soil or rock and the structure and with established principles of soil and rock mechanics.

10.6.1.2—Bearing Depth

Where the potential for scour, erosion or undermining exists, spread footings shall be located to bear below the maximum anticipated depth of scour, erosion, or undermining as specified in Article 2.6.4.4.

C10.6.1.2

Consideration should be given to the use of either a geotextile or graded granular filter material to reduce the susceptibility of fine grained material piping into rip rap or open-graded granular foundation material.

10.6.2—Service Limit State Design

10.6.2.1—General

Service limit state design of spread footings shall include evaluation of total and differential settlement and overall stability. Overall stability of a footing shall be evaluated where one or more of the following conditions exist:

- Horizontal or inclined loads are present,
- The foundation is placed on embankment,
- The footing is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

C10.6.2.1

The design of spread footings is frequently controlled by movement at the service limit state. It is therefore usually advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

10.6.2.2—Tolerable Movements

The requirements of Article 10.5.2.1 shall apply.

10.6.2.3—Loads

Immediate settlement shall be determined using load combination Service I, as specified in Table 3.4.1-1. Time-dependent settlements in cohesive soils should be determined using only the permanent loads, i.e., transient loads should not be considered.

C10.6.2.3

The type of load or the load characteristics may have a significant effect on spread footing deformation. The following factors should be considered in the estimation of footing deformation:

- The ratio of sustained load to total load,
- The duration of sustained loads, and
- The time interval over which settlement or lateral displacement occurs.

The consolidation settlements in cohesive soils are time-dependent; consequently, transient loads have negligible effect. However, in cohesionless soils where the permeability is sufficiently high, elastic deformation of the supporting soil due to transient load can take place. Because deformation in cohesionless soils often takes place during construction while the loads are being applied, it can be accommodated by the structure to an extent, depending on the type of structure and construction method.

Deformation in cohesionless, or granular, soils often occurs as soon as loads are applied. As a consequence, settlements due to transient loads may be significant in cohesionless soils, and they should be included in settlement analyses.

10.6.2.4—Settlement Analyses

10.6.2.4.1—General

Foundation settlements should be estimated using computational methods based on the results of laboratory or insitu testing, or both. The soil parameters

C10.6.2.4.1

Elastic, or immediate, settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. The magnitude of elastic settlement is used in the computations should be chosen to reflect the loading history of the ground, the construction sequence, and the effects of soil layering.

Both total and differential settlements, including time dependant effects, shall be considered.

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$$S_t = S_e + S_c + S_s \tag{10.6.2.4.1-1}$$

where:

S_e = elastic settlement (ft)

S_c = primary consolidation settlement (ft)

S_s = secondary settlement (ft)

estimated as a function of the applied stress beneath a footing or embankment. Elastic settlement is usually small and neglected in design, but where settlement is critical, it is the most important deformation consideration in cohesionless soil deposits and for footings bearing on rock. For footings located on over-consolidated clays, the magnitude of elastic settlement is not necessarily small and should be checked.

In a nearly saturated or saturated cohesive soil, the pore water pressure initially carries the applied stress. As pore water is forced from the voids in the soil by the applied load, the load is transferred to the soil skeleton. Consolidation settlement is the gradual compression of the soil skeleton as the pore water is forced from the voids in the soil. Consolidation settlement is the most important deformation consideration in cohesive soil deposits that possess sufficient strength to safely support a spread footing. While consolidation settlement can occur in saturated cohesionless soils, the consolidation occurs quickly and is normally not distinguishable from the elastic settlement.

Secondary settlement, or creep, occurs as a result of the plastic deformation of the soil skeleton under a constant effective stress. Secondary settlement is of principal concern in highly plastic or organic soil deposits. Such deposits are normally so obviously weak and soft as to preclude consideration of bearing a spread footing on such materials.

The principal deformation component for footings on rock is elastic settlement, unless the rock or included discontinuities exhibit noticeable time-dependent behavior.

The effects of the zone of stress influence, or vertical stress distribution, beneath a footing shall be considered in estimating the settlement of the footing.

For guidance on vertical stress distribution for complex footing geometries, see Poulos and Davis (1974) or Lambe and Whitman (1969).

Spread footings bearing on a layered profile consisting of a combination of cohesive soil, cohesionless soil and/or rock shall be evaluated using an appropriate settlement estimation procedure for each layer within the zone of influence of induced stress beneath the footing.

Some methods used for estimating settlement of footings on sand include an integral method to account for the effects of vertical stress increase variations. For guidance regarding application of these procedures, see Gifford et al. (1987).

The distribution of vertical stress increase below circular or square and long rectangular footings, i.e., where $L > 5B$, may be estimated using Figure 10.6.2.4.1-1.

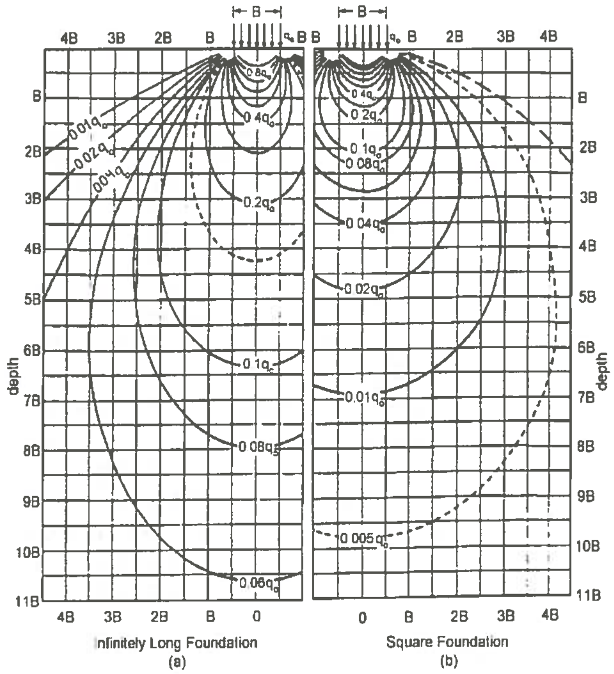

Figure 10.6.2.4.1-1—Boussinesq Vertical Stress Contours for Continuous and Square Footings Modified after Sowers (1979)

10.6.2.4.2—Settlement of Footings on Cohesionless Soils

The settlement of spread footings bearing on cohesionless soil deposits shall be estimated as a function of effective footing width and shall consider the effects of footing geometry and soil and rock layering with depth.

Settlements of footings on cohesionless soils shall be estimated using elastic theory or empirical procedures.

C10.6.2.4.2

Although methods are recommended for the determination of settlement of cohesionless soils, experience has indicated that settlements can vary considerably in a construction site, and this variation may not be predicted by conventional calculations.

Settlements of cohesionless soils occur rapidly, essentially as soon as the foundation is loaded. Therefore, the total settlement under the service loads may not be as important as the incremental settlement between intermediate load stages. For example, the total and differential settlement due to loads applied by columns and cross beams is generally less important than the total and differential settlements due to girder placement and casting of continuous concrete decks.

Generally conservative settlement estimates may be obtained using the elastic half-space procedure or the empirical method by Hough. Additional information regarding the accuracy of the methods described herein is provided in Gifford et al. (1987) and Kimmerling (2002). This information, in combination with local experience and engineering judgment, should be used when determining the estimated settlement for a structure foundation, as there may be cases, such as attempting to build a structure grade high to account for the estimated settlement, when overestimating the settlement magnitude could be problematic.

Details of other procedures can be found in textbooks and engineering manuals, including:

- Terzaghi and Peck (1967)
- Sowers (1979)
- U.S. Department of the Navy (1982)
- D'Appolonia (Gifford et al., 1987)—This method includes consideration for over-consolidated sands.
- Tomlinson (1986)
- Gifford et al. (1987)

For general guidance regarding the estimation of elastic settlement of footings on sand, see Gifford et al. (1987) and Kimmerling (2002).

The elastic half-space method assumes the footing is flexible and is supported on a homogeneous soil of infinite depth. The elastic settlement of spread footings, in feet, by the elastic half-space method shall be estimated as:

$$S_e = \frac{\left[q_o\left(1-\nu^2\right)\sqrt{A'}\right]}{144\ E_s\beta_z} \qquad (10.6.2.4.2\text{-}1)$$

where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of in situ or laboratory tests (ksi)

β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

ν = Poisson's Ratio, taken as specified in Article 10.4.6.3 if direct measurements of ν are not available from the results of in situ or laboratory tests (dim)

Unless E_s varies significantly with depth, E_s should be determined at a depth of about 1/2 to 2/3 of B below the footing, where B is the footing width. If the soil modulus varies significantly with depth, a weighted average value of E_s should be used.

Table 10.6.2.4.2-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

The stress distributions used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth. The settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footings, respectively. The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

Spread footings of the dimensions normally used for bridges are generally assumed to be rigid, although the actual performance will be somewhere between perfectly rigid and perfectly flexible, even for relatively thick concrete footings, due to stress redistribution and concrete creep.

The accuracy of settlement estimates using elastic theory are strongly affected by the selection of soil modulus and the inherent assumptions of infinite elastic half space. Accurate estimates of soil moduli are difficult to obtain because the analyses are based on only a single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent footings.

For footings with eccentric loads, the area, A', should be computed based on reduced footing dimensions as specified in Article 10.6.1.3.

Estimation of spread footing settlement on cohesionless soils by the empirical Hough method shall be determined using Eqs. 10.6.2.4.2-2 and 10.6.2.4.2-3.

The Hough method was developed for normally consolidated cohesionless soils.

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

C10.6.2.6.1

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf) Ordinary Range	Bearing Resistance (ksf) Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3 f'_c$.

10.6.3—Strength Limit State Design

10.6.3.1—Bearing Resistance of Soil

10.6.3.1.1—General

Bearing resistance of spread footings shall be determined based on the highest anticipated position of groundwater level at the footing location.

The factored resistance, q_R, at the strength limit state shall be taken as:

$$q_R = \varphi_b \, q_n \qquad (10.6.3.1.1\text{-}1)$$

where:

φ_b = resistance factor specified in Article 10.5.5.2.2

q_n = nominal bearing resistance (ksf)

Where loads are eccentric, the effective footing dimensions, L' and B', as specified in Article 10.6.1.3, shall be used instead of the overall dimensions L and B in all equations, tables, and figures pertaining to bearing resistance.

C10.6.3.1.1

The bearing resistance of footings on soil should be evaluated using soil shear strength parameters that are representative of the soil shear strength under the loading conditions being analyzed. The bearing resistance of footings supported on granular soils should be evaluated for both permanent dead loading conditions and short-duration live loading conditions using effective stress methods of analysis and drained soil shear strength parameters. The bearing resistance of footings supported on cohesive soils should be evaluated for short-duration live loading conditions using total stress methods of analysis and undrained soil shear strength parameters. In addition, the bearing resistance of footings supported on cohesive soils, which could soften and lose strength with time, should be evaluated for permanent dead loading conditions using effective stress methods of analysis and drained soil shear strength parameters.

The position of the groundwater table can significantly influence the bearing resistance of soils through its effect on shear strength and unit weight of the foundation soils. In general, the submergence of soils will reduce the effective shear strength of cohesionless (or granular) materials, as well as the long-term (or drained) shear strength of cohesive (clayey) soils. Moreover, the effective unit weights of submerged soils are about half of those for the same soils under dry conditions. Thus, submergence may lead to a significant reduction in the bearing resistance provided by the foundation soils, and it is essential that the bearing resistance analyses be carried out under the assumption of the highest groundwater table expected within the service life of the structure.

Footings with inclined bases should be avoided wherever possible. Where use of an inclined footing base cannot be avoided, the nominal bearing resistance determined in accordance with the provisions herein should be further reduced using accepted corrections for inclined footing bases in Munfakh, et al. (2001).

Because the effective dimensions will vary slightly for each limit state under consideration, strict adherence to this provision will require re-computation of the nominal bearing resistance at each limit state.

Further, some of the equations for the bearing resistance modification factors based on L and B were not necessarily or specifically developed with the intention that effective dimensions be used. The designer should ensure that appropriate values of L and B are used, and that effective footing dimensions L' and B' are used appropriately.

Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

10.6.3.1.2—Theoretical Estimation

10.6.3.1.2a—Basic Formulation

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \; BN_{\gamma m} C_{w\gamma} \qquad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \qquad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \qquad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \qquad (10.6.3.1.2a\text{-}4)$$

where:

c = cohesion, taken as undrained shear strength (ksf)

C10.6.3.1.2a

The bearing resistance formulation provided in Eqs. 10.6.3.1.2a-1 though 10.6.3.1.2a-4 is the complete formulation as described in the Munfakh, et al. (2001). However, in practice, not all of the factors included in these equations have been routinely used.

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

γ = total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

D_f = footing embedment depth (ft)

B = footing width (ft)

$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

s_c, s_γ, s_q = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)

d_q = correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4 (dim)

i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

For $\phi_f = 0$:

$$i_c = 1 - (nH/cBLN_c) \qquad (10.6.3.1.2a\text{-}5)$$

For $\phi_f > 0$:

$$i_c = i_q - [(1 - i_q)/(N_q - 1)] \qquad (10.6.3.1.2a\text{-}6)$$

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi_f)}\right]^n \qquad (10.6.3.1.2a\text{-}7)$$

Most geotechnical engineers nationwide have not used the load inclination factors. This is due, in part, to the lack of knowledge of the vertical and horizontal loads at the time of geotechnical explorations and preparation of bearing resistance recommendations.

Furthermore, the basis of the load inclination factors computed by Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8 is a combination of bearing resistance theory and small scale load tests on 1 in. wide plates on London Clay and Ham River Sand (Meyerhof, 1953). Therefore, the factors do not take into consideration the effects of depth of embedment. Meyerhof further showed that for footings with a depth of embedment ratio of $D_f/B = 1$, the effects of load inclination on bearing resistance are relatively small. The theoretical formulation of load inclination factors were further examined by Brinch-Hansen (*1970*), with additional modification by Vesic (*1973*) into the form provided in Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8.

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi_f)}\right]^{(n+1)} \quad (10.6.3.1.2a\text{-}8)$$

$$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta \quad (10.6.3.1.2a\text{-}9)$$

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

It should further be noted that the resistance factors provided in Article 10.5.5.2.2 were derived for vertical loads. The applicability of these resistance factors to design of footings resisting inclined load combinations is not currently known. The combination of the resistance factors and the load inclination factors may be overly conservative for footings with an embedment of approximately $D_f/B = 1$ or deeper because the load inclination factors were derived for footings without embedment.

In practice, therefore, for footings with modest embedment, consideration may be given to omission of the load inclination factors.

Figure C10.6.3.1.2a-1 shows the convention for determining the θ angle in Eq. 10.6.3.1.2a-9.

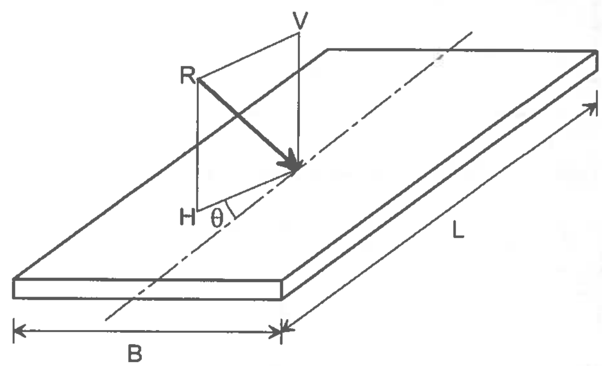

Figure C10.6.3.1.2a-1—Inclined Loading Conventions

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The parent information from which Table 10.6.3.1.2a-4 was developed covered the indicated range of friction angle, ϕ_f. Information beyond the range indicated is not available at this time.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \qquad (10.6.3.1.2b\text{-}1)$$

C10.6.3.1.2b

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

The design procedures for foundations in rock have been developed using the RMR, rock mass rating system. Classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

10.6.3.2.2—Semiempirical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geomechanics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures.

The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

C10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on rock mass quality measured in terms of *RMR* system, the nominal bearing resistance of a rock mass varies from a small fraction to six times the unconfined compressive strength of intact rock core samples.

10.6.3.2.3—Analytic Method

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be considered.

C10.6.3.2.3

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing resistance for each of the failure modes can be found in Kulhawy and Goodman (1987), Goodman (1989), and Sowers (1979).

10.6.3.2.4—Load Test

Where appropriate, load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

- One-third of the corresponding footing dimension, B or L, for footings on soils, or 0.45 of the corresponding footing dimensions B or L, for footings on rock.

C10.6.3.3

A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of $B/3$ were comparable to those of ASD with an eccentricity of $B/6$. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of $B/2$ would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Articl... and a comparison of those resistance factors t... Allowable Stress Design practice, e.g., ... (2002), is provided in Allen (2005).

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

Scour design for the design flood must... requirement that the factored foundation resi... scour is greater than the factored load dete... the scoured soil removed. The resistance fa... those used in the Strength Limit State, witho...

10.5.5.2.2—Spread Footings

C10.5.5.2.2

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength L...

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

Sliding
for Abutments and Retaining Walls

PROBLEM STATEMENT & OBJECTIVE

Determine the sliding coefficient of friction resistance factor for the strength limit state for the abutment footings on bedrock and retaining wall footings on medium dense granular material.

EXECUTIVE SUMMARY

The sliding coefficient of friction, tanδ, for the cast-in-place concrete footings for the abutments is 0.7
The sliding coefficient of friction, tanδ, for the cast-in-place concrete footings for the retaining wall is 0.55
The resistance factor for sliding for both the abutments and retaining walls is 0.8

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

ASSUMPTIONS

1. Footings will be cast-in-place concrete.
2. Bearing material is bedrock for the abutments and medium dense granular backfill for the retaining walls.

CALCULATIONS

Use Table 3.11.5.3-1 for sliding coefficient of friciton for cast-in-place concrete on bedrock or soil:

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

Use 0.7 for cast-in-place concrete footings on bedrock and 0.55 for cast-in-place concrete footings on medium dense granular soil.

HALEY ALDRICH	CALCULATIONS	File No.	41107-200
		Sheet	2 of 2
Client	CLD Consulting Engineers, Inc.	Date	21-Jul-17
Project	Bridge No. 114, US Route 7 over Neshobe River, Brandon, Vermont	Computed by	MMH
Subject	Sliding Resistance	Checked by	JGD

Use Table 10.5.5.2.2-1 for resistance factor for sliding for cast-in-place concrete on bedrock or soil:

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength L

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

Use 0.8 for cast-in-place concrete footings on soil or bedrock.

CONCLUSIONS AND RECOMMENDATIONS

The sliding coefficient of friction, tanδ, for the cast-in-place concrete footings for the abutments is 0.7
The sliding coefficient of friction, tanδ, for the cast-in-place concrete footings for the retaining wall is 0.55
The resistance factor for sliding for both the abutments and retaining walls is 0.8

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THE VOICE OF TRANSPORTATION

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Part II: Sections 7—Index

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Publication Code: LRFDUS-7

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Articl... and a comparison of those resistance factors t... Allowable Stress Design practice, e.g., ... (2002), is provided in Allen (2005).

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

Scour design for the design flood must ... requirement that the factored foundation resi... scour is greater than the factored load dete... the scoured soil removed. The resistance fac... those used in the Strength Limit State, witho...

10.5.5.2.2—Spread Footings

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

C10.5.5.2.2

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength L...

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Global Stability
for East Retaining Wall

PROBLEM STATEMENT & OBJECTIVE

Calculate the factor of safety against global stability failure for the east retaining wall.

EXECUTIVE SUMMARY

The factor of safety for under the service load is 3.25
The factor of safety for under an extreme event is 2.35

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
2. Slide version 7.0 by RocScience.

AVAILABLE INFORMATION

1. Plan set titled "Proposed Improvement Bridge Project, Town of Brandon, County of Rutland, US Route 7 (Principal Aterial) Bridge No. 114"
2. Haley & Aldrich boring logs from August 2015.
3. Loading provided by CLD on 19 June 2017.

ASSUMPTIONS

1. Retaining wall is modeled as infinite strength with no weight.
2. Bedrock is modeled as infinite strength with unit weight of 150 pcf.
2. A service load of 313 ksf will be applied at the bottom of the retaining wall (service load provided by CLD for east wall).
3. A sidewalk live load of 250 psf will be applied.
4. A traffic live load of 250 psf will be applied.
5. A factor of safety of 1.5 is considered acceptable for the Service I load combination based on AASHTO LRFD Section 11.6.2.3
6. The factor of safety for circular failures including the retaining wall structure will be observed.
7. Groundwater modeled at 408.8 (Q100 level).

SOIL PROPERTIES

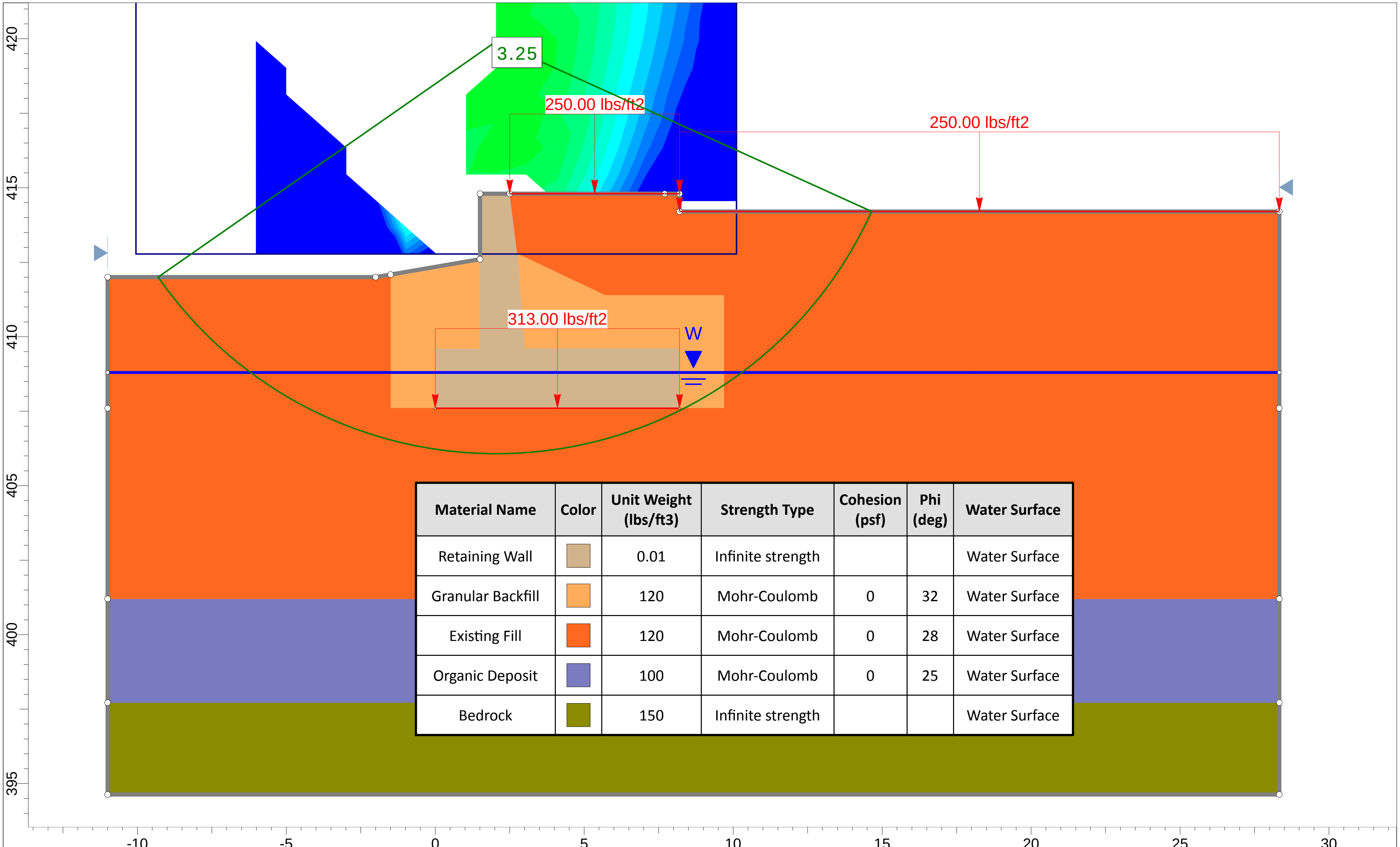
Material	Unit Weight (lbs/ft^3)	Friction Angle (degrees)
Existing Fill	120	28
Granular Backfill	120	32
Organic Deposits	100	25

SOIL PROFILE

Boring	Elevation of Top of Organics	Thickness of Organic Deposit	Elevation of Top of Bedrock
HA-B1	--	--	--
HA-B1A	--	--	--
HA-B1B	400.7	3	397.7
HA-B1C	--	--	--
HA-B2	409.5	3.5	406

RESULTS

Case	Factor of Safety
Service	3.25
Extreme Event	2.35

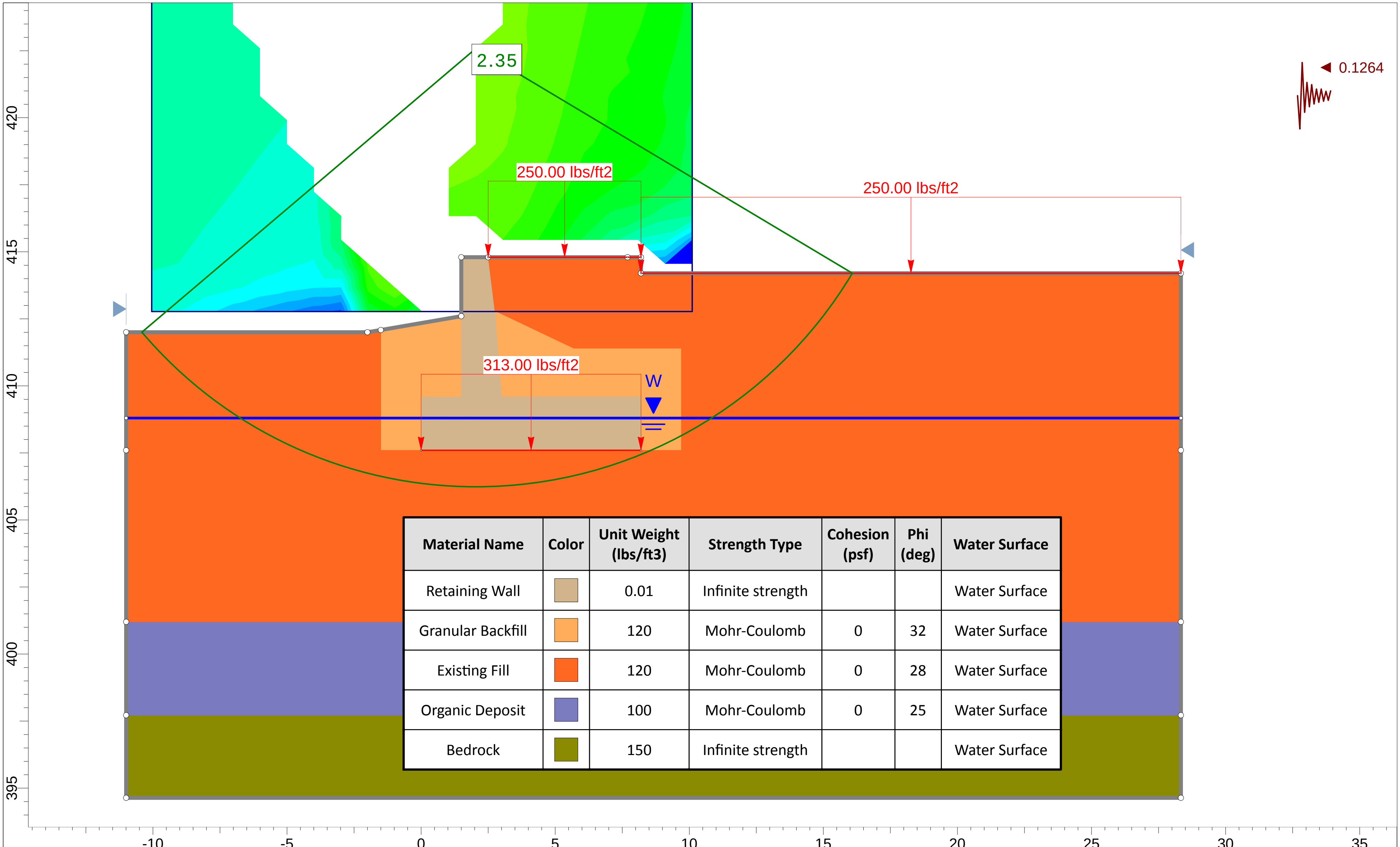


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Retaining Wall		0.01	Infinite strength			Water Surface
Granular Backfill		120	Mohr-Coulomb	0	32	Water Surface
Existing Fill		120	Mohr-Coulomb	0	28	Water Surface
Organic Deposit		100	Mohr-Coulomb	0	25	Water Surface
Bedrock		150	Infinite strength			Water Surface

HALEY ALDRICH

SLIDEINTERPRET 7.018

Project	East Retaing Wall		
Analysis Description	Global Stability		
Drawn By	MMH	*Company*	Haley & Aldrich
Date	June 2017	*File Name*	2017-0622-HAI-East Retaining Wall-Static-D1.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Retaining Wall		0.01	Infinite strength			Water Surface
Granular Backfill		120	Mohr-Coulomb	0	32	Water Surface
Existing Fill		120	Mohr-Coulomb	0	28	Water Surface
Organic Deposit		100	Mohr-Coulomb	0	25	Water Surface
Bedrock		150	Infinite strength			Water Surface

HALEY ALDRICH

SLIDEINTERPRET 7.018

Project	East Retaing Wall		
Analysis Description	Global Stability		
Drawn By	MMH	*Company*	Haley & Aldrich
Date	June 2017	*File Name*	2017-0622-HAI-East Retaining Wall-Seismic.slim

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11.6.2—Movement and Stability at the Service Limit State

11.6.2.1—Abutments

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 shall apply as applicable.

11.6.2.2—Conventional Retaining Walls

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 apply as applicable.

C11.6.2.2

For a conventional reinforced concrete retaining wall, experience suggests that differential wall settlements on the order of 1 in 500 to 1 in 1,000 may overstress the wall.

11.6.2.3—Overall Stability

The overall stability of the retaining wall, retained slope and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. The overall stability of temporary cut slopes to facilitate construction shall also be evaluated. Special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft deposits.

The evaluation of overall stability of earth slopes with or without a foundation unit should be investigated at the Service I Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, ϕ, may be taken as:

- Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element... 0.75
- Where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element 0.65

C11.6.2.3

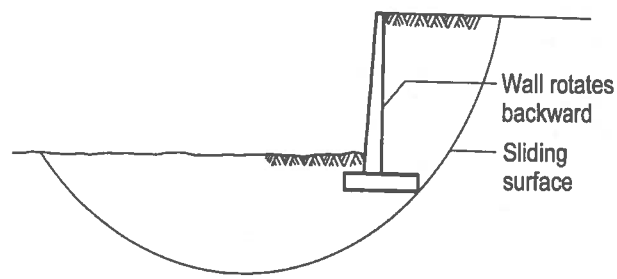

Figure C11.6.2.3-1—Retaining Wall Overall Stability Failure

Figure C11.6.2.3-1 shows a retaining wall overall stability failure. Overall stability is a slope stability issue, and, therefore, is considered a service limit state check.

The Modified Bishop, simplified Janbu or Spencer methods of analysis may be used.

Soft soil deposits may be subject to consolidation and/or lateral flow which could result in unacceptable long-term settlements or horizontal movements.

With regard to selection of a resistance factor for evaluation of overall stability of walls, examples of structural elements supported by a wall that may justify the use of the 0.65 resistance factor include a bridge or pipe arch foundation, a building foundation, a pipeline, a critical utility, or another retaining wall. If the structural element is located beyond the failure surface for external stability behind the wall illustrated conceptually in Figure 11.10.2-1, or if the wall does not support a structural element, a resistance factor of 0.75 may be used.

Available slope stability programs produce a single factor of safety, *FS*. The specified resistance factors are essentially the inverse of the *FS* that should be targeted in the slope stability program.

Seismic Site Class

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using available subsurface, SPT N or shear wave velocity (Vs) information.

EXECUTIVE SUMMARY

A Site Class D is recommended.

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (2012 Interim Revisions).
2. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
3. United States Geological Survey Website.

AVAILABLE INFORMATION

1. Boring logs dated 04-08-2015 to 06-08-2015 by New England Boring. Borings HA-B1 to HA-B2.
2. Other: e.g., Vs or geophysical data report, lab data, geotech reports, etc.
3. Elevations reference the NAVD 88 datum.

ASSUMPTIONS

1. Where SPT N, Vs or su data was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N, Vs or su for the extended profile was then assumed based on the available information.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.4.2.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A
Average shear wave velocity for the upper 100 ft of the soil profile:

$$\overline{V_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of *i* th soil (ft/s).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

PROCEDURE

Method B
Average standard penetration test (SPT) for the upper 100 ft of the soil profile:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th soil layer not to exceed 100 ft (blows/ft).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

Method C
Average standard penetration test (SPT) for the cohesionless layers in the upper 100 ft of the soil profile:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th cohesionless soil layer (blows/ft).
d_i = thickness of *i* th cohesionless soil layer (ft).
m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and m.

Average undrained shear strength for the cohesive layers in the upper 100 ft of the soil profile:

$$\bar{s}_u = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

where

s_{ui} = undrained shear strength of *i* th cohesive soil layer (psf), not to exceed 5000 psf
d_i = thickness of *i* th cohesive soil layer (ft).
k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and k.

Based on the available information, Method B will be used for the seismic Site Class evaluation.

SITE CLASS DEFINITIONS

(Table from AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (with 2012 Interim Revisions)).

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec < $\bar{v}_s$ < 5000 ft/sec
C	Very dense soil and soil rock with 1200 ft/sec < $\bar{v}_s$ < 2500 ft/sec, or with either $\bar{N}$ > 50 blows/ft or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1200 ft/sec, or with either 15 blows/ft < $\bar{N}$ < 50 blows/ft or 1.0 ksf < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/sec, or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40%, and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific ground motion response evaluations, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay, where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H > 120 ft)

Exceptions:

Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

HALEY ALDRICH	CALCULATIONS	File No.	41107-200
		Sheet	4 of 6
Client	CLD Consulting Engineers, Inc.	Date	6-Jun-17
Project	Two-Span Stone Arches Bridge No. 114, Brandon, Vermont	Computed by	ASC
Subject	Seismic Site Class	Checked by	MMH

CALCULATIONS - METHOD B

Exploration ID: HA-B1
Ground Surface El.: 414.7

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
S1	2	412.7	Fill	3.0	21	0.143
S2	4	410.7	Fill	2.0	53	0.038
S3	6	408.7	Fill	2.0	44	0.045
S4	8	406.7	Fill	2.0	30	0.067
S5	10	404.7	Fill	3.5	25	0.140
S6	15	399.7	Organics	4.5	2	2.250
		414.7	Bedrock	83.0	100	0.830
			Totals =	100.0		3.513

N-bar (blows/ft) = 28.5
Site Class = D

Note: Considered HA-B1, HA-B1A, and HA-B1B.

	CALCULATIONS	File No.	41107-200
		Sheet	5 of 6
Client	CLD Consulting Engineers, Inc.	Date	6-Jun-17
Project	Two-Span Stone Arches Bridge No. 114, Brandon, Vermont	Computed by	ASC
Subject	Seismic Site Class	Checked by	MMH

CALCULATIONS - METHOD B

Exploration ID:	HA-B2
Ground Surface El.:	414.5

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
S1	1.5	413	Fill	2.5	18	0.139
S2	3.5	411	Fill	2.5	14	0.179
S3	6	408.5	Forest Mat	1.5	2	0.750
S4	7.5	407	Organics	2.0	2	1.000
		414.5	Bedrock	91.5	100	0.915
			Totals =	100.0		2.982

N-bar (blows/ft) = 33.5
Site Class = D

HALEY ALDRICH	CALCULATIONS	File No.	41107-200
		Sheet	6 of 6
Client	CLD Consulting Engineers, Inc.	Date	6-Jun-17
Project	Two-Span Stone Arches Bridge No. 114, Brandon, Vermont	Computed by	ASC
Subject	Seismic Site Class	Checked by	MMH

RESULTS SUMMARY

Boring Number	Parameter	Average Value	Site Class
HA-B1	SPT-N	28.5	D
HA-B2	SPT-N	33.5	D

From the USGS Website:

S_1 =	0.049	g
S_S =	0.173	g
PGA =	0.079	g
F_v =	2.4	
F_a =	1.6	
F_{PGA} =	1.6	
S_{D1} =	0.1176	g
S_{DS} =	0.2768	g
A_s =	0.1264	g

CONCLUSIONS & RECOMMENDATIONS

Recommend a Site Class D.

LIMITATIONS

2011

AASHTO Guide Specifications for

LRFD Seismic Bridge Design

2nd Edition

2012 Interim Revisions

AMERICAN ASSOCIATION OF
STATE HIGHWAY AND
TRANSPORTATION OFFICIALS

AASHTO
THE VOICE OF TRANSPORTATION

Published by the
American Association of State Highway and Transportation Officials

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- The bridge is considered critical or essential according to Article 4.2.2, for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.2 is desired.

If the site is located within 6 mi of a known active fault capable of producing a magnitude 5 earthquake and near fault effects are not modeled in the development of national ground motion maps, directivity and directionality effects should be considered as described in Article 3.4.3.1 and its commentary.

seismic activity for each earthquake source zone located in the vicinity of the site. The result of a PSHA is a relationship of the mean annual rate of exceedance of the ground motion parameter of interest with each potential seismic source considered. See Kramer (1996) for further discussions of the types and methods used to conduct DSHAs and PSHAs.

- *Site-Specific Ground Motion Response Analysis*: A site-specific ground response analysis is used to determine the influence of local ground conditions on the design ground motions. The analysis is generally based on the assumption of a vertically propagating shear wave though more complex analyses can be conducted if warranted. A site-specific ground motion response analysis is typically used to evaluate the influence of "non-standard" soil profiles on ground response to the seismic hazard level. Site-specific ground motion response analyses may also be used to assess the effects of pore-water pressure build-up on ground response, vertical motions resulting from compression wave propagation, laterally non-uniform soil conditions, incoherence, and the spatial variation of ground motions.

In these provisions, an active fault is defined as a near-surface or shallow fault whose location is known or can reasonably be inferred and which has exhibited evidence of displacement in Holocene (or recent) time (in the past 11,000 yr, approximately). Active fault locations can be found from maps showing active faults prepared by state geological agencies or the U.S. Geological Survey. The manner in which an active fault is used in a DSHA and a PSHA is different and should be appropriately treated when conducting each type of analysis.

Article C3.4.3 describes near-fault ground-motion effects that are not included in national ground-motion mapping and could potentially increase the response of some bridges. Normally, site-specific evaluation of these effects would be considered only for essential or very critical bridges.

3.4.1—Design Spectra Based on General Procedure

If a site-specific hazard analysis is not conducted, design response spectra shall be constructed using response spectral accelerations taken from national ground motion maps described in this Article and site factors described in Article 3.4.2. The construction of the response spectra shall follow the procedures described below and illustrated in Figure 3.4.1-1.

C3.4.1

National ground-motion maps are based on probabilistic national ground motion mapping conducted by the U.S. Geological Survey (USGS) having a seven percent chance of exceedance in 75 yr. Values for PGA, S_s and S_1 can be obtained from the maps in these Guide Specifications or from the USGS seismic parameters CD-ROM accompanying these Guide Specifications. The CD-ROM provides the coefficients by the latitude and longitude of the bridge site, or by ZIP code for the site. Use of the latitude and longitude is the preferred procedure when using the CD-ROM.

An error has been identified in the Spectral Response Accelerations S_{DS} and S_{D1} results produced by the CD-ROM software. Specifically, the A_s value is erroneously calculated as A_s=F_a PGA. Although the corrected value for A_s is presented in the tabulated Design Spectrum table, designers should be aware of this error until the problem is corrected. The software error will likely have negligible effects on bridge analysis results because:

- F_{pga} is approximately equal to F_a,
- A_s is properly calculated and displayed in the tabulated design spectra, and
- Bridges have fundamental periods greater than the effected period range ($T < T_o$).

In lieu of using national ground motion maps referenced in these Guide Specifications, ground motion response spectra may be constructed on the basis of approved state ground motion maps. To be accepted, the development of state maps should conform to the following:

- The definition of design ground motion return period or probability of exceedance should equal or exceed those described in Article 3.2.
- Ground motion maps should be based on a detailed analysis demonstrated to lead to a quantification of ground motion, at a regional scale, that is as accurate or more so as achieved in the national maps. The analysis should include characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source models, ground motion models, and parameter values used in the analysis; detailed documentation of map development; and detailed peer review as deemed appropriate by the Owner. The peer review process should preferably include individuals from the USGS, other organizations, or both who have expertise in developing probabilistic seismic hazard maps on a regional basis.

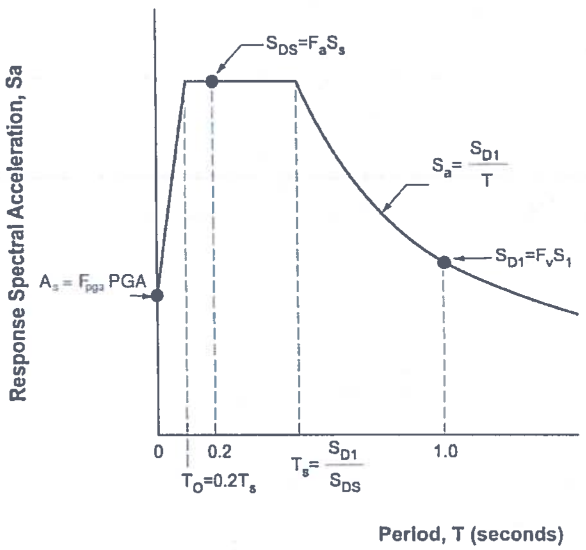


Figure 3.4.1-1—Design Response Spectrum, Construction Using Three-Point Method

Design earthquake response spectral acceleration coefficients for the acceleration coefficient, A_S, the short period acceleration coefficient, S_{DS}, and at the 1-sec period acceleration coefficient, S_{D1}, shall be determined from Eqs. 3.4.1-1 through 3.4.1-3, respectively:

$$A_s = F_{pga}PGA \quad (3.4.1\text{-}1)$$

$$S_{DS} = F_a S_s \quad (3.4.1\text{-}2)$$

$$S_{D1} = F_v S_1 \quad (3.4.1\text{-}3)$$

where:

F_{pga} = site coefficient for peak ground acceleration defined in Article 3.4.2.3

PGA = peak horizontal ground acceleration coefficient on Class B rock

F_a = site coefficient for 0.2-sec period spectral acceleration specified in Article 3.4.2.3

The design response spectrum includes the short-period transition from acceleration coefficient, A_s, to the peak response region, S_{DS}, unlike the AASHTO *Standard Specifications for Highway Bridges*, Division I-A. This transition is effective for all modes, including the fundamental vibration modes. Use of the peak response down to zero period is felt to be overly conservative, particularly for displacement-based designs. The use of R_d (see Article 4.3.3) to magnify displacements in the short-period range also offsets the reductions in conservatism when using the transition from A_s to S_{DS}.

For periods exceeding approximately 3 sec, depending on the seismic environment, Eq. 3.4.1-8 may be conservative because the ground motions may be approaching the constant spectral displacement range for which S_a decays with period as $1/T^2$. The long-period transition to constant displacement has been incorporated into recent maps used in the building industry (e.g., *International Building Code* (ICC, 2006). However, the constant displacement portion of the response spectrum has not been included herein. Typical structures for which this region would apply are either long-span non-conventional structures and thus

S_s = 0.2-sec period spectral acceleration coefficient on Class B rock

F_v = site coefficient for 1.0-sec period spectral acceleration specified in Article 3.4.2.3

S_1 = 1.0-sec period spectral acceleration coefficient on Class B rock

Linear interpolation shall be used to determine the ground motion parameters PGA, S_S, and S_1 for sites located between contour lines or between a contour line and a local maximum or minimum.

The design response spectrum curve shall be developed as follows and as indicated in Figure 3.4.1-1:

- For periods less than or equal to T_o, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = (S_{DS} - A_s)\frac{T}{T_o} + A_s \quad (3.4.1\text{-}4)$$

in which:

$$T_o = 0.2T_S \quad (3.4.1\text{-}5)$$

$$T_S = \frac{S_{D1}}{S_{DS}} \quad (3.4.1\text{-}6)$$

where:

A_s = acceleration coefficient

S_{D1} = design spectral acceleration coefficient at 1.0-sec period

S_{DS} = design spectral acceleration coefficient at 0.2-sec period

T = period of vibration (sec)

- For periods greater than or equal to T_o and less than or equal to T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = S_{DS} \quad (3.4.1\text{-}7)$$

- For periods greater than T_S, the design response spectral acceleration coefficient, S_a, shall be defined as follows:

$$S_a = \frac{S_{D1}}{T} \quad (3.4.1\text{-}8)$$

Response spectra constructed using maps and procedures described in Article 3.4.1 are for a damping ratio of five percent and do not include near field ground motion adjustments. See Article 3.4.3.1 for near-field adjustments.

beyond the scope of these Guide Specifications, or they are structures that would warrant a site-specific response analysis. In the latter case, constant displacement attributes of the response spectrum should be considered during the development of the site-specific ground motion hazard. The long-period transition identified in IBC (2006) is for a design earthquake with a two percent probability of exceedance in 50 yr (i.e., 2475-yr return period), and therefore should not be used.

The coefficient obtained for the USGS/AASHTO Seismic Hazard Maps are based on a uniform seismic hazard. The probability that a coefficient will not be exceeded at a given location during a 75-yr period is estimated to be about 93 percent, i.e., seven percent probability of exceedance. The use of a 75-yr interval matches the design life prescribed by the *AASHTO LRFD Bridge Design Specifications*.

It can be shown that an event with a seven percent probability of exceedance in 75 yr has a return period of about 1,000 yr. This earthquake is called the design earthquake.

The value of the spectral acceleration parameters (A_s, S_{DS}, and S_{D1}) need not use more than two decimal places.

3.4.2—Site Effects on Ground Motions

The generalized site classes and site factors described in this Article shall be used with the general procedure for constructing response spectra described in Article 3.4.1. Site-specific analysis of soil response effects shall be conducted where required by Article 3.4 and in accordance with the requirements in Article 3.4.3 and Table 3.4.2.1-1—Site Class Definitions.

If geological conditions at the abutments and intermediate piers result in different soil classification, then the site factors used to develop the design response spectrum may be determined based upon the site-specific procedures outlined in Article 3.4.3. In lieu of the site-specific procedures and under guidance from the geotechnical engineer, the design response spectrum should be determined by constructing a response spectrum for individual abutments, piers, or groups of piers and then developing a single spectrum based on the higher spectral acceleration coefficient at each period, i.e., an envelope of the spectra.

C3.4.2

The behavior of a bridge during an earthquake is strongly related to the soil conditions at the site. Soils can amplify or deamplify ground motions originating in the underlying rock. The amount of amplification can be a factor of two or more. The extent of amplification or deamplification is dependent on the profile of the soil types at the site and the intensity of shaking in the rock below. Sites are classified by types and profile for the purposes of defining the overall seismic hazard, which is quantified as the product of soil amplification or deamplification and intensity of shaking in the underlying rock.

The site classes and site factors described in this Article were originally recommended at a site response workshop in 1992 (Martin, ed., 1994). Subsequently, they were adopted in the seismic design criteria of Caltrans, the 1994 and 1997 editions of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1995, 1998), the 1997 *Uniform Building Code* (ICBO, 1997), and subsequently the *International Building Codes* (ICC, 2000, 2003, and 2006). The bases for the adopted site classes and site factors is described by Martin and Dobry (1994) and Rinne (1994).

Procedures described in this Article were originally developed for computing ground motions at the ground surface for relatively uniform site conditions. Depending on the site classification and the level of the ground motion, the motion at the surface will likely be different from the motion at depth. This creates some question as to the location of the motion to use in the bridge design. It is also possible that the soil conditions at the two abutments are different or they differ at the abutments and interior piers. An example would be where one abutment is on firm ground or rock and the other is on a loose fill. These variations are not always easily handled by simplified procedures described in this commentary. For critical bridges, it may be necessary to use more rigorous numerical modeling to represent these conditions. The decision to use more rigorous numerical modeling should be made after detailed discussion of the benefits and limitations of more rigorous modeling between the Bridge and Geotechnical Engineers and the Owner.

3.4.2.1—Site Class Definitions

The site shall be classified as one of the following classes given in Table 3.4.2-1. Procedures given in Article 3.4.2.2 shall be used to determine the average condition for varying profile conditions.

For preliminary design, Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated on the basis of shear wave velocities in similar competent rock with

C3.4.2.1

Steps for classifying a site (also see Table 3.4.2-1):

Step 1: Check the site against the three categories of Site Class F, requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.

Step 2: Categorize the site using one of the following three methods, with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified by the definitions in Article 3.4.2.2:

moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft, surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

The rock categories, Site Classes A and B, shall not be used if there is more than 100 ft of soil between the rock surface and the bottom of the spread footing or mat foundation.

PI shall be taken as the plasticity index specified in ASTM D 4318. The moisture content, *w*, shall be taken as the moisture content in percent specified in ASTM D 2216.

Method A: $\bar{v}_s$ for the top 100 ft ($\bar{v}_s$ method)

Method B: $\bar{N}$ for the top 100 ft ($\bar{N}$ method)

Method C: $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft and average $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft ($\bar{s}_u$ method)

The values $\bar{v}_s$, $\bar{N}_{ch}$, and $\bar{s}_u$ are averaged over the respective thickness of cohesionless and cohesive soil layers within the upper 100 ft. Refer to Article 3.4.2.2 for equations for calculating average parameter values for Methods A, B, and C. If Method C is used, the site class is determined as the softer site class resulting from the averaging to obtain $\bar{N}_{ch}$ and $\bar{s}_u$ (for example, if $\bar{N}_{ch}$ were equal to 20 blows/ft and $\bar{s}_u$ were equal to 800 psf, the site would classify as E in accordance with Table 3.4.2-1). Note that when using Method B, $\bar{N}$ values are for both cohesionless and cohesive soil layers within the upper 100 ft.

As described in Article C3.4.2.2, it may be appropriate in some cases to define the ground motion at depth, below a soft surficial layer, if the surficial layer would not significantly influence bridge response. In this case, the site class may be determined on the basis of the soil profile characteristics below the surficial layer.

Within Site Class F (soils requiring site-specific evaluation), one category has been deleted in these Guide Specifications from the four categories contained in the previously cited codes and documents. This category consists of soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible, weakly cemented soils. It was judged that special analyses for the purpose of refining site ground motion amplifications for these soils were too severe a requirement for ordinary bridge design because such analyses would require utilization of effective stress and strength-degrading nonlinear analyses that are difficult to conduct. Also, limited case-history data and analysis results indicate that liquefaction reduces spectral response rather than increases it, except at long periods in some cases. (e.g., $T = 1$ sec) Because of the general reduction in response spectral amplitudes due to liquefaction, the designer may wish to consider special analysis of site response for liquefiable soil sites to avoid excessive conservatism in assessing bridge inertia loads when liquefaction occurs.

Site-specific analyses are required for major or very important structures in some cases (Article 3.4), so that appropriate analysis techniques would be used for such structures. The deletion of liquefiable soils from Site Class F only affects the requirement to conduct site-specific analyses for the purpose of determining ground motion amplification through these soils. It is still required to evaluate liquefaction occurrence and its effect on a bridge as specified in Article 6.8.

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec < $\bar{v}_s$ < 5000 ft/sec
C	Very dense soil and soil rock with 1200 ft/sec < $\bar{v}_s$ < 2500 ft/sec, or with either $\bar{N}$ > 50 blows/ft or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1200 ft/sec, or with either 15 blows/ft < $\bar{N}$ < 50 blows/ft or 1.0 ksf < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/sec, or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40%, and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific ground motion response evaluations, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay, where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H > 120 ft)

Exceptions:

Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

3.4.2.2—Definitions of Site Class Parameters

The definitions presented below shall be taken to apply to the upper 100 ft of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft.

The average $\bar{v}_s$ for the site profile shall be taken as:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{3.4.2.2-1}$$

where:

$\sum_{i=1}^{n} d_i$ = thickness of upper soil layers = 100 ft

C3.4.2.2

If the site profile is particularly nonuniform, or if the average velocity computed in this manner does not appear reasonable, or if the project involves special design issues, it may be desirable to conduct shear wave velocity measurements. In all evaluations of site classification, the shear wave velocity should be viewed as the fundamental soil property, as this was used when conducting the original studies defining the site categories.

Use of Empirical v_{si} Relations: An alternative to applying Eqs. 3.4.2.2-2, 3.4.2.2-3, and 3.4.2.2-4 to obtain values for $\bar{N}$, $\bar{N}_{ch}$, and $\bar{s}_u$ is to convert the N values or s_u values into estimated shear wave velocities and then to apply Eq. 3.4.2.2-1. Procedures given in Kramer (1996) can be used for these conversions. The empirical equations identified in Kramer (1996) and in other references can involve significant uncertainty at a specific site, and this

d_i = thickness of ith soil layer (ft)

n = total number of distinctive soil layers in the upper 100 ft of the site profile below the bridge foundation

v_{si} = shear wave velocity of ith soil layer (ft/sec)

i = any one of the layers between 1 and n

$\bar{N}$ shall be taken as:

$$\bar{N} = \frac{\sum_{i=1} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad (3.4.2.2\text{-}2)$$

where:

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of ith soil layer not to exceed 100 ft (blows/ft).

$\bar{N}_{ch}$ shall be taken as:

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \quad (3.4.2.2\text{-}3)$$

where:

m = total number of cohesionless soil layers in the upper 100 ft of the site profile below the bridge foundation

$\bar{s}_u$ shall be taken as:

$$\bar{s}_u = \frac{\sum_{i=1}^{k} d_i}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \quad (3.4.2.2\text{-}4)$$

where:

k = total number of cohesive soil layers in the upper 100 ft of the site profile below the bridge foundation

s_{ui} = undrained shear strength of ith soil layer not to exceed 5 ksf

should be considered during the use of the empirical equations. The preferred approach is to calibrate the empirical procedure using in-situ velocity measurements when the empirical equations are to be used.

Depth of Motion Determination: For short bridges that involve a limited number of spans, the motion at the abutment will generally be the primary mechanism by which energy is transferred from the ground to the bridge superstructure. If the abutment is backed by an earth approach fill, the site classification should be determined at the base of the approach fill. The potential effects of the approach fill overburden pressure on the shear wave velocity of the soil should be accounted for in the determination of site classification.

For long bridges it may be necessary to determine the site classification at an interior pier. If this pier is supported on spread footings, then the motion computed at the ground surface is appropriate. However, if deep foundations (i.e., driven piles or drilled shafts) are used to support the pier, then the location of the motion will depend on the horizontal stiffness of the soil-cap system relative to the horizontal stiffness of the soil-pile system. If the pile cap is the stiffer of the two, then the motion should be defined at the pile cap. If the pile cap provides little horizontal stiffness or if there is no pile cap (i.e., pile extension), then the controlling motion will likely be at some depth below the ground surface. Typically this will be approximately 4 to 7 pile diameters below the pile cap or where a large change in soil stiffness occurs. The determination of this elevation requires considerable judgment and should be discussed by the geotechnical and bridge engineers.

For cases where the controlling motion is more appropriately specified at depth, site-specific ground response analyses can be conducted to establish ground motions at the point of fixity. This approach or alternatives to this approach should be used only with the Owner's approval.

3.4.2.3—Site Coefficients

Site coefficients for the peak ground acceleration F_{pga}, short-period range F_a, and for the long-period range F_v shall be taken as specified in Tables 3.4.2.3-1 and 3.4.2.3-2. Application of these coefficients to determine elastic seismic response coefficients of ground motion shall be as specified in Article 3.4.1.

C3.4.2.3

Site Class B (soft rock) is taken to be the reference site category for USGS and IBC ground motion site factors. Site Class B rock is therefore the site condition for which the site factor is 1.0. Site Classes A, C, D, and E have separate sets of site factors for zero-period (F_{pga}), the short-period range (F_a), and the long-period range (F_v), as indicated in

Tables 3.4.2.3-1 and 3.4.2.3-2. These site factors generally increase as the soil profile becomes softer (in going from Site Class A to E). Except for Site Class A (hard rock), the factors also decrease as the ground motion level increases, due to the strongly nonlinear behavior of soil. For Site Classes C, D, or E, these nonlinear site factors increase the ground motion more in areas having lower rock ground motions than in areas having higher rock ground motions.

Table 3.4.2.3-1—Values of F_{pga} and F_a as a Function of Site Class and Mapped Peak Ground Acceleration or Short-Period Spectral Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods				
	$PGA \leq 0.10$ $S_s \leq 0.25$	$PGA = 0.20$ $S_s = 0.50$	$PGA = 0.30$ $S_s = 0.75$	$PGA = 0.40$ $S_s = 1.00$	$PGA \geq 0.50$ $S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	[a]	[a]	[a]	[a]	[a]

Note: Use straight line interpolation for intermediate values of PGA and S_s, where PGA is the peak ground acceleration and S_s is the spectral acceleration coefficient at 0.2 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	[a]	[a]	[a]	[a]	[a]

Note: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration coefficient at 1.0 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

3.4.3—Response Spectra Based on Site-Specific Procedures

A site-specific procedure should be used to develop design response spectra of earthquake ground motions when required by Article 3.4 and may be performed for any site subject to the Owner's approval. The site-specific procedure can involve a site-specific hazard analysis, a site-specific ground motion response analysis, or both.

Unless otherwise approved by the Owner, where the response spectrum is developed using a site-specific hazard analysis, a site-specific ground motion response analysis, or both, the spectrum shall not be lower than two-thirds of the response spectrum at the ground surface determined using the general procedure of Article 3.4.1 adjusted by the site coefficients in Article 3.4.2.3 in the region of $0.5T_F$ to $2T_F$ of the spectrum, where T_F is the bridge fundamental period. For other analyses, such as liquefaction assessment and retaining wall design, the free-field acceleration at the ground surface should not be less than two-thirds of A_s determined from the general procedure.

C3.4.3

When estimating the minimum ground surface response spectrum using two-thirds of the response spectrum from the general procedure in Article 3.4.1 and the site coefficients in Article 3.4.2.3, there are no site coefficients for liquefiable sites or for sites that fall in Site Class F. No consensus currently exists regarding the appropriate site coefficients for these cases. Unless the Owner directs otherwise, the following approach should be used:

- For liquefiable sites, use the site coefficient based on soil conditions without any modifications for liquefaction. This approach is believed to be conservative for higher frequency motions, and the Owner may decide to use a minimum spectrum lower than the two-thirds value. However, when accepting a spectrum lower than two-thirds of the spectrum identified in the above discussions, the uncertainties in the analysis method should be carefully reviewed, particularly for longer periods (i.e., $T > 1.0$ sec) where increases in the spectral ordinate may occur. If a lower factor than two-thirds is being considered, it is suggested that an independent peer review of the results of the site-specific analyses be performed.
- For Site Class F locations, the recommended approach is to accept the results of a site-specific study subject to the concurrence of the Owner and an independent peer review panel. In previous guidance documents (ATC and MCEER, 2003), the suggestion was made to use a Site Class E site coefficient for Site Class F soils. This approach appears to be overly conservative and is not suggested.

3.4.3.1—Site-Specific Hazard Analysis

If the probabilistic seismic hazard analysis (PSHA) is used, the site-specific analysis shall be conducted in a manner to generate a uniform-hazard acceleration response spectrum considering a seven percent probability of exceedance in 75 yr for spectral values over the entire period range of interest. This analysis shall establish the following:

- The contributing seismic sources,
- An upper-bound earthquake magnitude for each source zone,
- Median attenuation relations for acceleration response spectral values and their associated standard deviations,
- A magnitude-recurrence relation for each source zone, and
- A magnitude fault-rupture length or source area relation for each contributing fault or source area.

C3.4.3.1

The intent in conducting a site-specific hazard study is to develop ground motions that are more accurate for the local seismic and site conditions than can be determined from national ground motion maps and the procedure of Article 3.4.1. Accordingly, such studies should be comprehensive and incorporate current scientific interpretations at a regional scale. Because there are typically scientifically credible alternatives for models and parameter values used to characterize seismic sources and ground-motion attenuation, it is important to incorporate these uncertainties formally in a site-specific hazard analysis. Examples of these uncertainties include seismic source location, extent, and geometry; maximum earthquake magnitude; earthquake recurrence rate; and ground-motion attenuation relationship.

Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly affect bridge response. Spatial variations include differences between seismic wave arrival times at bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

3.5—SELECTION OF SEISMIC DESIGN CATEGORY (SDC)

Each bridge shall be assigned to one of four seismic design categories (SDCs), A through D, based on the 1-sec period design spectral acceleration for the design earthquake (S_{D1}, refer to Article 3.4.1) as shown in Table 3.5-1.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.

C3.5

The seismic hazard level is defined as a function of the magnitude of the ground surface shaking as expressed by F_vS_1. However, other factors may affect the SDC selected. For example, if the soil is liquefiable and lateral spreading or slope failure can occur, SDC D should be selected. For assessment of existing structures, the Designer should also consider using SDC D regardless of the magnitude of A_s, even when significant lateral soil movement is not expected, if the structure is particularly weak with regard to its ability to resist the forces and displacements that could be caused by the liquefaction (see Article C6.8).

The SDC reflects the variation in seismic risk across the country and is used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

Value of $S_{D1} = F_vS_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

The requirements for each of the proposed SCDs shall be taken as shown in Figure 3.5-1 and described below. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6, respectively, and shall also meet minimum support length requirements of Article 4.12.

- SDC A

    a. No identification of ERS according to Article 3.3

    b. No demand analysis

    c. No implicit capacity check needed

    d. No capacity design required

    e. Minimum detailing requirements for support length, superstructure/substructure connection design force, and column transverse steel

    f. No liquefaction evaluation required

- SDC B

    g. Identification of ERS according to Article 3.3 should be considered

    h. Demand analysis

    i. Implicit capacity check required (displacement, P-Δ, support length)

    j. Capacity design should be considered for column shear; capacity checks should be considered to avoid weak links in the ERS

    k. SDC B level of detailing

    l. Liquefaction check should be considered for certain conditions

- SDC C

    m. Identification of ERS

    n. Demand analysis

    o. Implicit capacity check required (displacement, P-Δ, support length)

    p. Capacity design required including column shear requirement

    q. SDC C level of detailing

    r. Liquefaction evaluation required

- SDC D

    s. Identification of ERS

    t. Demand analysis

    u. Displacement capacity required using pushover analysis (check P-Δ and support length)

    v. Capacity design required including column shear requirement

    w. SDC D level of detailing

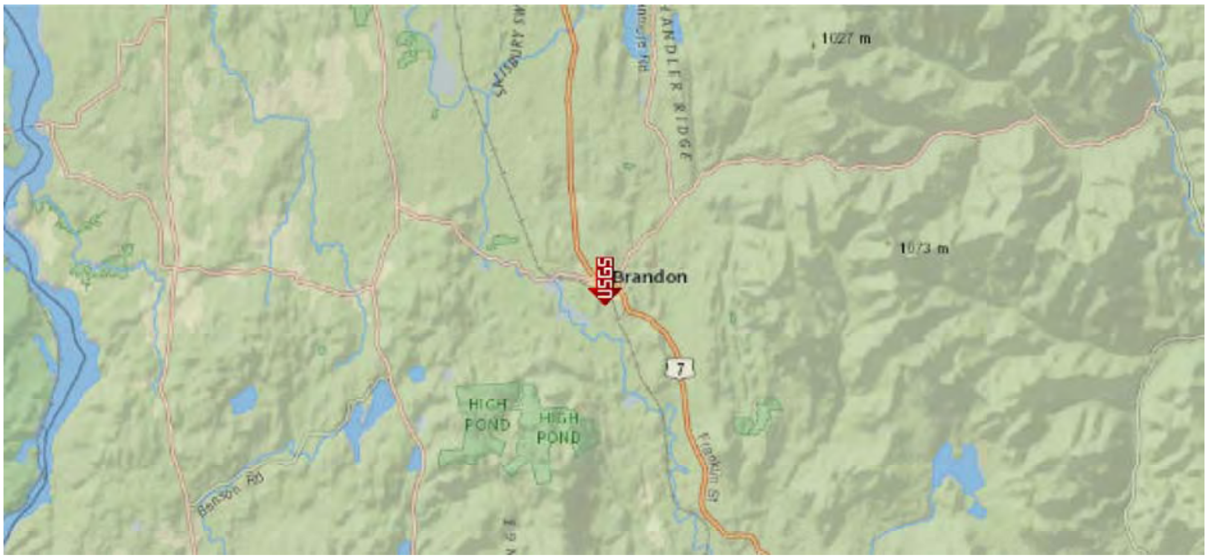
    x. Liquefaction evaluation required

Article 3.5

Design Maps Summary Report

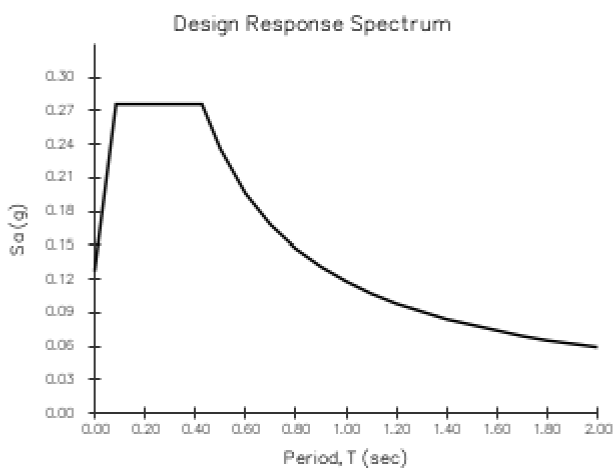
User–Specified Input

Report Title	Brandon, Vermont Wed June 14, 2017 19:18:31 UTC
Building Code Reference Document	2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (which utilizes USGS hazard data available in 2002)
Site Coordinates	43.79956°N, 73.08917°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

PGA =	0.079 g	**A_s =**	0.127 g
S_S =	0.173 g	**S_{DS} =**	0.276 g
S_1 =	0.049 g	**S_{D1} =**	0.118 g



USGS Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (43.79956°N, 73.08917°W)

Site Class D – "Stiff Soil"

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 [1]	PGA = 0.079 g
From Figure 3.4.1-3 [2]	S_S = 0.173 g
From Figure 3.4.1-4 [3]	S_1 = 0.049 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1–1 Site Class Definitions

<table>
<tr><th>SITE CLASS</th><th>SOIL PROFILE NAME</th><th>Soil shear wave velocity, $\overline{v}_s$, (ft/s)</th><th>Standard penetration resistance, $\overline{N}$</th><th>Soil undrained shear strength, $\overline{s}_u$, (psf)</th></tr>
<tr><td>A</td><td>Hard rock</td><td>$\overline{v}_s > 5{,}000$</td><td>N/A</td><td>N/A</td></tr>
<tr><td>B</td><td>Rock</td><td>$2{,}500 < \overline{v}_s \le 5{,}000$</td><td>N/A</td><td>N/A</td></tr>
<tr><td>C</td><td>Very dense soil and soft rock</td><td>$1{,}200 < \overline{v}_s \le 2{,}500$</td><td>$\overline{N} > 50$</td><td>>2,000 psf</td></tr>
<tr><td>D</td><td>Stiff soil profile</td><td>$600 \le \overline{v}_s < 1{,}200$</td><td>$15 \le \overline{N} \le 50$</td><td>1,000 to 2,000 psf</td></tr>
<tr><td>E</td><td>Stiff soil profile</td><td>$\overline{v}_s < 600$</td><td>$\overline{N} < 15$</td><td><1,000 psf</td></tr>
<tr><td>E</td><td>—</td><td colspan="3">Any profile with more than 10 ft of soil having the characteristics:
1. Plasticity index $PI > 20$,
2. Moisture content $w \ge 40\%$, and
3. Undrained shear strength $\overline{s}_u < 500$ psf</td></tr>
<tr><td>F</td><td>—</td><td colspan="3">Any profile containing soils having one or more of the following characteristics:
1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.
2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil)
3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$)
4. Very thick soft/medium stiff clays ($H > 120$ feet)</td></tr>
</table>

For SI: 1ft/s = 0.3048 m/s 1lb/ft^2 = 0.0479 kN/m^2

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.079 g, F_{PGA} = 1.600

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	S_S ≤ 0.25	S_S = 0.50	S_S = 0.75	S_S = 1.00	S_S ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.173 g, F_a = 1.600

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_1

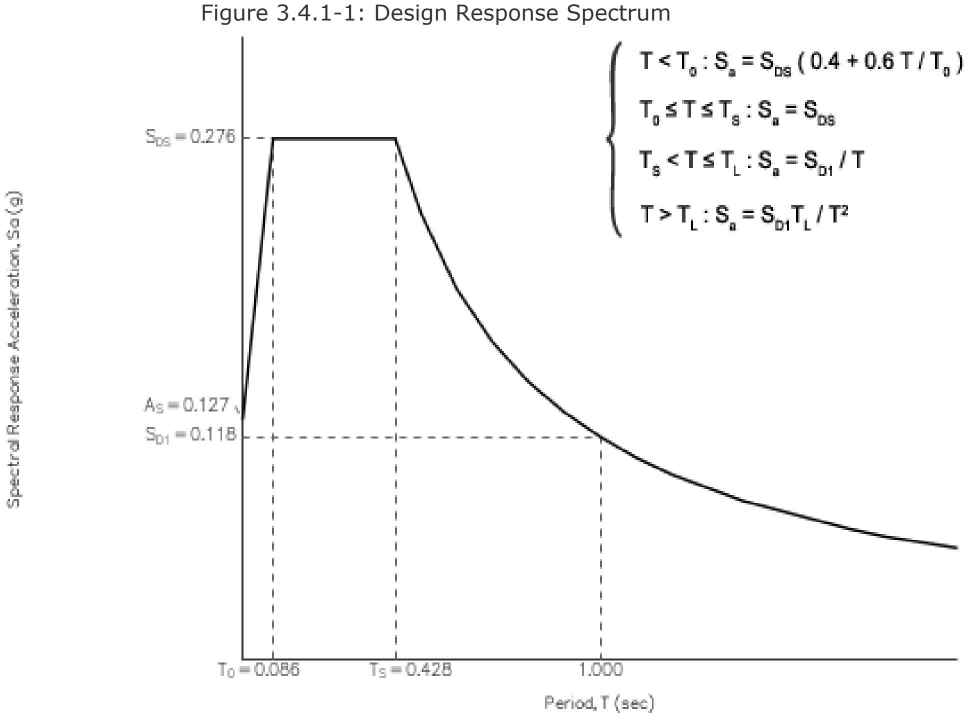
For Site Class = D and S_1 = 0.049 g, F_v = 2.400

Equation (3.4.1-1): $A_S = F_{PGA}\ PGA = 1.600 \times 0.079 = 0.127$ g

Equation (3.4.1-2): $S_{DS} = F_a\ S_S = 1.600 \times 0.173 = 0.276$ g

Equation (3.4.1-3): $S_{D1} = F_v\ S_1 = 2.400 \times 0.049 = 0.118$ g

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

For S_{D1} = 0.118 g, Seismic Design Category = A

Seismic Design Category ≡ “the design category in accordance with Table 3.5-1” = A

References

1. *Figure 3.4.1-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf
2. *Figure 3.4.1-3*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf
3. *Figure 3.4.1-4*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf